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Journal of the

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Proceedings of the American Society of Civil Engineers

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November, 1959

Papers

(Over)

Journal of the
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CAVITATION DAMAGE OF ROUGHENED CONCRETE SURFACES[a]

Donald Colgate,[1] M. ASCE

SUMMARY

The paper discusses an exploratory laboratory study concerning the eval-
ıation of the cavitation potential of various types of roughened concrete sur-
aces. Methods of evaluating the data determined by the laboratory study, and
neans of presenting the results for use by design and field engineers, are
presented.

The vigorous campaign against cavitation erosion of hydraulic installations continues space. Technical personnel in the hydraulic laboratories of our universities, private concerns, and those operated by the Government have clearly defined what cavitation is, and have, in the past 30 years, advanced several divergent and unrelated plausible explanations as to just how this harmless appearing vapor cloud can inflict such unbelievable damage to construction materials. The subsequent discussions that are encouraged when these different opinions are aired before all who are interested in the problem have led to the remarkable advances made thus far in the methods of protecting our hydraulic installations.

The common construction material most readily damaged by cavitation is concrete. Since this material is so relatively inexpensive, and so handily shaped to the desires of the designer, its use will continue to be wide and varied. However, due to its susceptibility to cavitation erosion, the specifications regulating the configuration of, and the very texture of, concrete surfaces which will be subjected to high velocity flow have become extremely stringent. This trend is proper.

At Grand Coulee Dam the forms on the spillway face bulged outward permitting the concrete to "hump" about 3 inches in at least one of the 5-foot lifts. An examination of the spillway face after about 9 seasons of operation

Note: Discussion open until April 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2241 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 11, No. HY 11, November, 1959.

a. Presented at the October, 1956, ASCE Convention in Pittsburgh, Pa.

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disclosed that the concrete downstream from the hump had been damaged by cavitation (Fig. 1). In this instance the designers had specified that the spillway section conform in all respects to the "lines, grades, and dimensions" shown on the drawings. In practice, some deviation from true lines is to be expected, but the permissible limits of such deviation to prevent damaging cavitation have not been accurately determined, therefore, in light of the Grand Coulee experiences, extremely rigid specifications are now adhered to.

It is common practice to evaluate extensive damage where cavitation is a factor by explaining that the initial erosion is caused by cavitation, and that the large eroded areas downstream are the result of jet action. From a practical viewpoint, the mechanics of the total damage is of secondary importance- the damaged area is repaired and the cause of the cavitation ascertained, if possible, and corrected. It is likely, however, that damaging cavitation forms on the roughened surfaces and continues its destruction far downstream from that area affected by the initial cavitation pocket. Jet action contributes to the damage by washing away loosened particles. This does not mean that jet action is to be discounted as a damaging agent.

The relative destructiveness of a jet and of cavitation has been inadvertently demonstrated on several occasions. Fresh mortar has been splashed onto a concrete surface and allowed to remain and set up; later the surface has been subjected to high velocity flow. The mortar lump, merely sticking to the surface, will remain even though exposed to the full force of the jet, but the cavitation caused by its presence will erode away the concrete downstream.

During the building of Davis Dam on the Colorado River, discharges were made into the side of the spillway bucket. Debris of varied origin was washed

Fig. 1. Spillway, Grand Coulee Dam, Showing "humped" Concrete Surface and Cavitation Erosion.

about until quite an area was scoured, some places to a depth of 3 inches or more below the finished surface (Figs. 2 and 3). When the bucket was de-watered and the scoured area inspected, the question arose as to whether or not these roughened surfaces would induce cavitation. Plaster molds were made of several different damaged areas, and concrete casts made from these molds, so that the cavitation potential of the various surfaces might be evaluated in the Bureau of Reclamation Hydraulic Laboratory. However, spillway releases were imminent and insufficient time remained for a laboratory study, therefore, the repair of all the scoured area to the original contours was authorized and performed.

The problem concerning the severity of surface roughness which might be tolerated remained to be solved. Since the casts of the roughened surfaces from Davis Dam were on hand, the Bureau initiated a limited exploratory program to devise a means of evaluating various roughened surfaces as regards their cavitation potential. The program envisioned eventual classification of

Fig. 2. 1-foot-square Plaster Mold of Eroded Surface of Davis Dam (Specimen No. 2).

surface texture for specifications for new installations for repairing roughened surfaces.

A laboratory testing apparatus was designed and built to accommodate the Davis Dam specimen. The apparatus (Fig. 4) permitted a stream 6 inches wide and 3 inches deep to pass over the test surface. The test section top was made of a transparent material so that the presence of cavitation could be determined visually (Fig. 5). During the preliminary test run it was determined that, under certain conditions, cavitation did form on the protruding aggregate (Fig. 6).

The roughened surfaces used in this study have been referred to as Specimen No. 2 and Specimen No. 3. The average exposed aggregate of specimen No. 2 extended about 3/4 inch above the lowest point of the roughened surface, and that of Specimen No. 3 about 1/4 inch. The laboratory pumps were capable of producing velocities up to about 85 fps through the test section, and the discharge piping was so arranged that the pressure head could be lowered to about 17 feet of water below atmospheric pressure. The measurements made were (a) the discharge, (b) pressures at various points on the viewing window, and (c) single leg Pitot tube pressures to calculate the velocities at various distances above the specimen.

In the analysis the data are applicable to either closed conduits or open channels. Although the laboratory study was made in a closed conduit rectangular in cross section, the computations were made using circular pipe formulae. These approximations appear to be permissible since the boundary

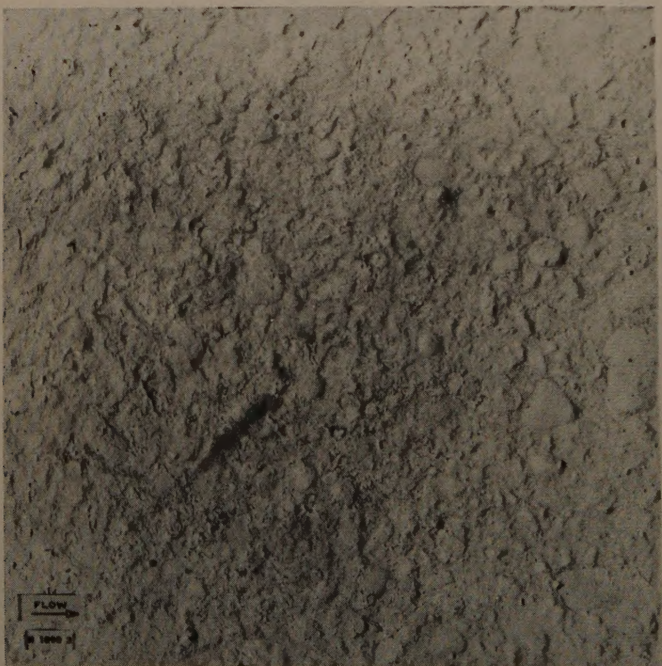

Figure 3. 1-foot-square plaster mold of eroded surface at Davi
Dam (Specimen No. 3).

hear, and the stream velocity near the boundary, are affected by a given surace in the same manner for either closed conduit or open channel flow. The omputations regarding the velocity distribution in the stream above the surace recognize that the Prandtl universal logarithmic velocity-distribution aw for pipes applies equally well for an infinitely wide open channel.

The analysis considered that the tested surface was a full scale specimen, nd that the boundary shear, or shear velocity, of the surface could be deternined directly from measurements made in the laboratory apparatus. A plot n semilog paper of the velocity profile perpendicular to the test specimen roduced a straight line for the elements of flow affected only by the roughend surface (Fig. 7). From such a plot the shear velocity for each specimen as determined from the Karman-Prandtl equation for rough surfaces:

$$\frac{V}{\sqrt{\frac{\tau_o}{\rho}}} = 5.75 \log_{10} \frac{Y}{K} + 8.5 \tag{1}$$

here:

Y = distance from the specimen

V = velocity at Y

K = a constant related to the boundary roughness

$\sqrt{\frac{\tau_o}{\rho}}$ = shear velocity

Substituting values for two points (V_1, Y_1) and (V_2, Y_2) and solving siultaneously, the shear velocity was determined:

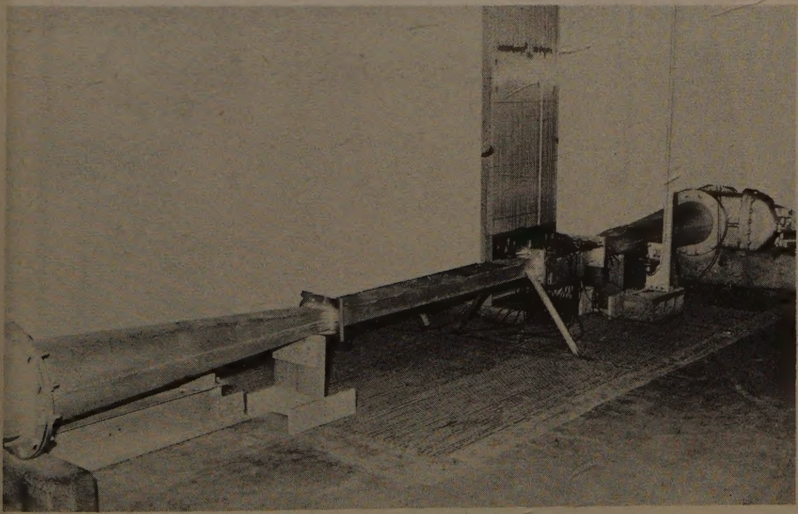

Figure 4. View of laboratory test apparatus showing approach and downstream piping.

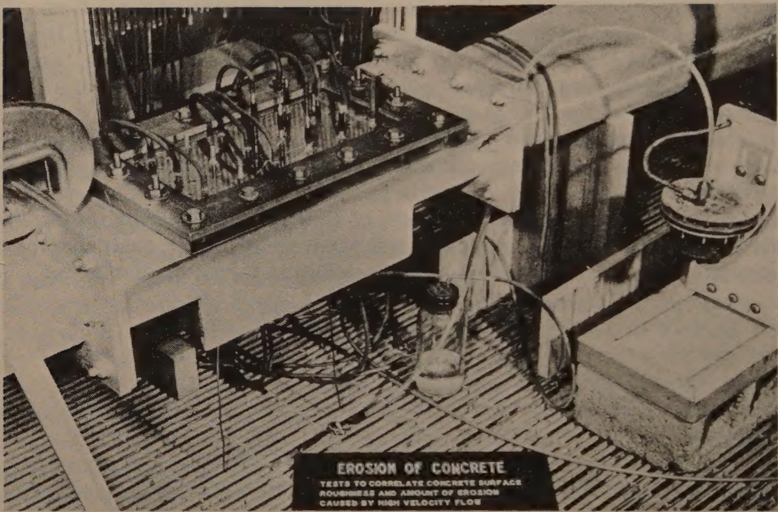

Figure 5. Closeup of test section showing transparent viewing window and pressure taps.

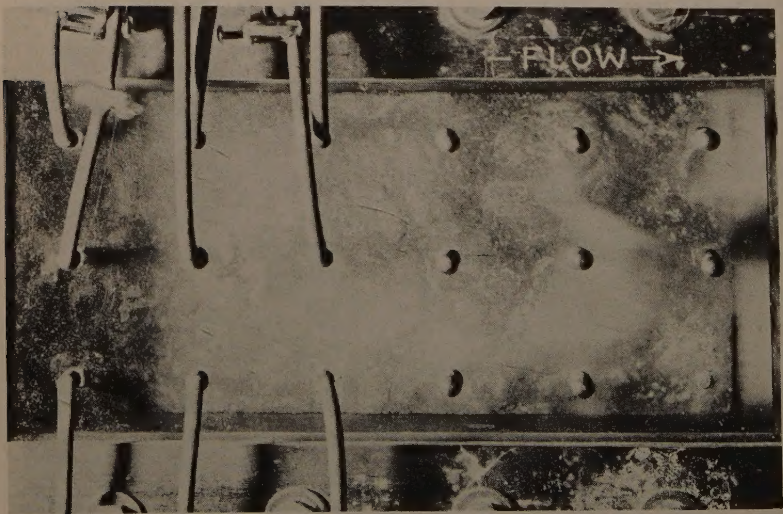

Figure 6. Visible cavitation cloud on Specimen No. 2. Head on the viewing window = +14', $\sqrt{\frac{T_0}{\rho}} = 5.4$.

$$\sqrt{\frac{\tau_o}{\rho}} = \frac{V_2 - V_1}{5.75 \log_{10} \frac{Y_2}{Y_1}} \tag{2}$$

r the boundary shear:

$$\tau_o = (0.0586) \left(\frac{V_1 - V_2}{\log_{10} \frac{Y_1}{Y_2}} \right)^2 \tag{3}$$

The discharge through the test apparatus and the pressure at the specimen ere adjusted to produce a small, but visible cavitation cloud, and the veloc- y profile measured above the specimen. This procedure was followed for everal combinations of velocity and pressure. A plot was then made showing ne relationship betweeen the shear velocity and the pressure on the viewing indow (Fig. 8).

This curve is readily obtainable from the test apparatus, but the possibil- y is remote that flowing water in a field installation will have been in con- act with the roughened surface long enough to establish uniform turbulent ow, and an accurate determination of the shear velocity would be virtually npossible.

A study of various velocity profiles (Fig. 9) reveals that, for the same verage velocity, the velocity near the boundary is appreciably greater for a mooth surface than for a rough one. If the approach to the roughened area as a reasonably smooth surface sufficiently long to establish uniform flow, ne average velocity of the approaching stream necessary to cause cavitation

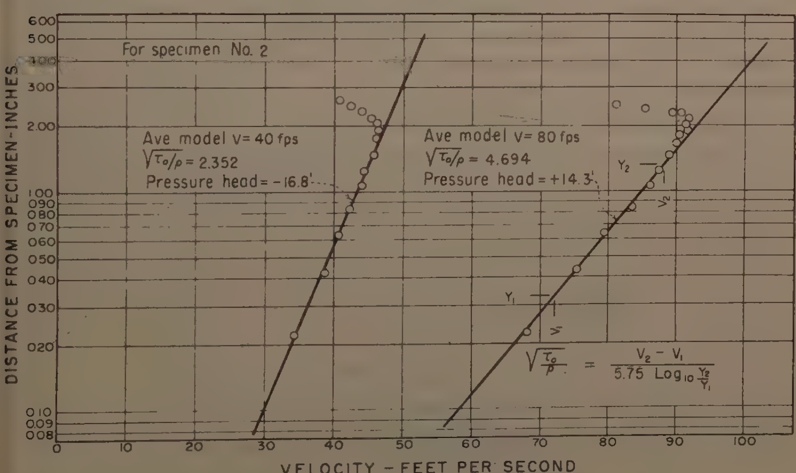

Figure 7. Graph showing representative velocity distribution for Specimen No. 2.

on the rough surface would be less than that computed from the shear velocity curve (Fig. 7). Since the velocity near the boundary is the one which would attack the roughened surface, it is the one which must be computed to ascertain whether cavitation would exist on the damaged or roughened area.

For illustration, assume that a roughened surface of Specimen No. 2 texture existed on the floor of a channel with water flowing 5 feet deep, and that

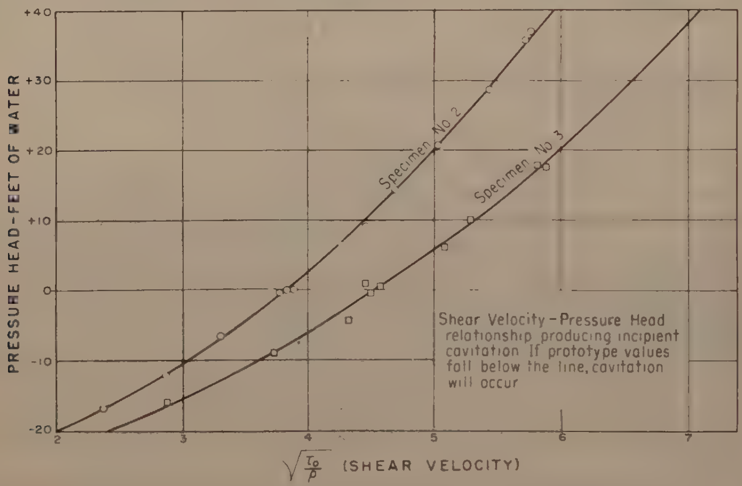

Figure 8. Relationship between stream depth (or pressure head) and shear velocity, feet per second.

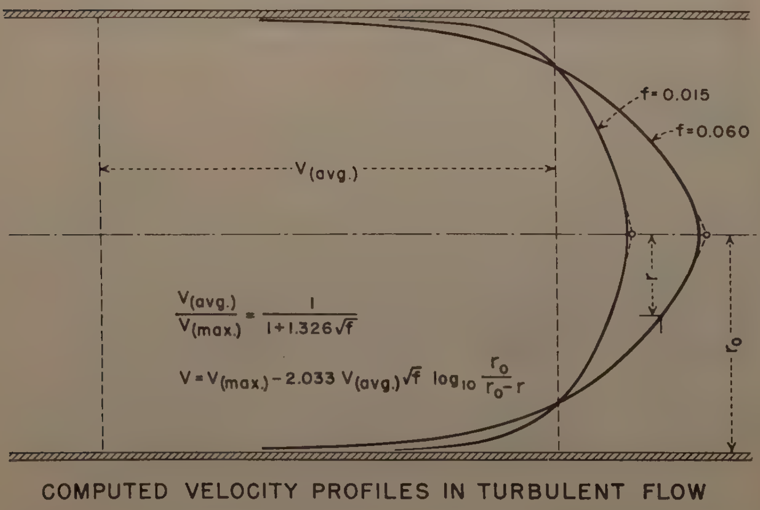

Figure 9. Computed velocity profiles for smooth and rough surfaces in turbulent flow.

is surface prevailed for a considerable distance upstream. From Fig. 8 e shear velocity at which cavitation would occur would be 4.2 fps. Consider e velocity about 0.3 inch from the mean surface to be critical. This veloc-y may be computed (for this surface) from the relation:

$$V_{0.3}^2 = 110\ (H_p + H_B)$$

(determined from the average of the results of the study)

here:

$V_{0.3}$ = velocity 0.3 inch from the mean surface

H_p = pressure head or depth

H_B = barometric pressure in feet of water

o:

$$V_{0.3} = \sqrt{110\ (5 + 27)} = 59.3 \text{ fps}$$

he average velocity exists at:

$Y = (0.37)(5) = 1.85$ feet from the bottom (from Vanoni, Velocity Distribution in Open Channels, Civil Engineering, June 1941, p. 357)

ubstituting in Equation (2):

$$4.2 = \frac{V_{avg} - 59.3}{5.75 \log_{10} \frac{1.85}{0.025}}$$

$V_{avg} = 104.4$ fps

(average stream velocity at which cavitation will occur on Specimen No. 2 with rough approach surface)

Now assume that the approach channel to the roughened surface is smooth- and has about one-quarter the boundary shear of the roughened area.

en:

$$2.1 = \frac{V_{avg} - 59.3}{5.75 \log_{10} \frac{1.85}{0.025}}$$

$V_{avg} = 81.9$ fps

(average stream velocity at which cavitation will occur on Specimen No. 2 with a relatively smooth approach sur-face)

In the case of flow with a blunt velocity profile, the average velocity can considered to exist near enough to the boundary to affect the roughened

area. If the average velocity is critical, then cavitation will occur on Specimen No. 2 at

$$V_{avg} = 59.3 \text{ fps}$$

Comparing the approach conditions and average stream velocities for incipient cavitation to exist on the roughened surface in the illustration:

Approach conditions	Average stream velocity
Specimen No. 2 texture	104.4 fps
Relatively smooth approach	81.9 fps
Blunt velocity profile	59.3 fps

From the data obtained in the laboratory a chart was made plotting the average velocity near the roughened surface versus depth of flow (or pressure head) for the condition of incipient cavitation for the two tested surfaces (Fig. 10).* A family of curves of this type for various surface textures would be extremely valuable to the designer, or to the field engineer, since the values for the chart can be readily computed from known flow conditions.

On the basis of average velocity and stream depth, the flow conditions at Davis Dam would induce cavitation on the damaged surfaces (Fig. 10).

This exploratory study on cavitation of roughened surfaces demonstrated that a simple laboratory test can be made to evaluate the cavitation potential of any surface. There is a need for the collection, evaluation, and classification of information from the field concerning roughened surface problems, and further laboratory studies must be made so that criteria may be established to enable the designer or field engineer to state with confidence that a given surface texture will or will not induce cavitation.

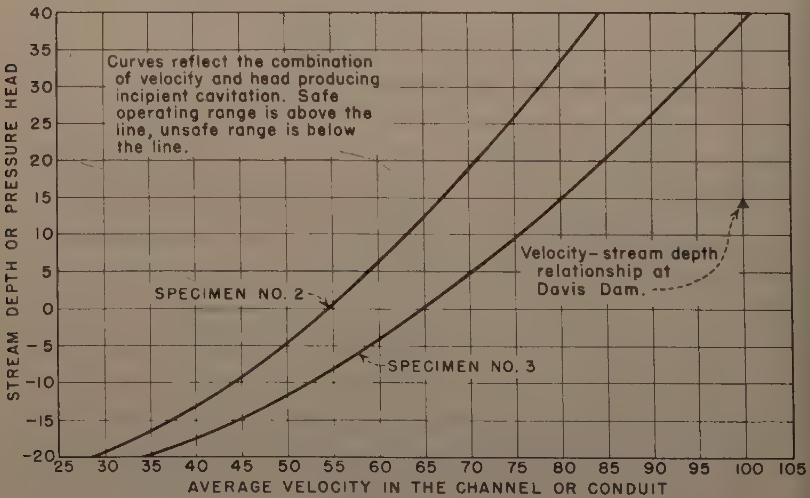

Figure 10. Relationship between stream depth (or pressure head) and average stream velocity, feet per second.

*Fig. 10 shows the most rigorous conditious: that of a stream flowing with a blunt velocity profile. If a stream has a partially or fully developed boundary layer, and the average stream velocity is used, the chart will show pressures which are well into the "safe" range.

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MOUNTAIN CHANNEL TREATMENT IN LOS ANGELES COUNTY[a]

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ABSTRACT

This paper presents a brief history and the current status of channel treat-ent projects in the San Gabriel Mountains of Los Angeles County which are ing constructed to lessen the financial burden of flood protection and for the nservation of water.

Mountain channel treatment works in Los Angeles County are undertaken accomplish three main objectives. These are: (1) to protect the inhabi-its and property below the watersheds from damage by flood-borne rock d debris, (2) to lessen the financial burden of maintaining the large number debris-retaining and flood-regulating structures that have been constructed a part of the flood control program, and (3) to aid in conserving water for e in the developed area below.

The population of the Los Angeles Coastal Plain has increased from 300,000 to 5,600,000 persons in the past 10 years and is projected to reach 500,000 by 1970. Keeping pace with this growth has been the increase in using and industrial development. The impression one gets from an aerial ew of the broad alluvial cones that have been built up during the past ousands of years immediately below the watersheds forming the perimeter the Coastal Plain is that here is where most of this growth is taking place. e San Gabriel Mountains, which form the northern boundary of this peri-eter, extend from the San Fernando Valley in the west, eastward for 55 les to and beyond the Los Angeles-San Bernardino County line (see location ap, Fig. 1). With the exception of a few watersheds in the nearby Verdugo ountains, watershed treatment efforts have been and will continue to be

te: Discussion open until April 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2242 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 11, No. HY 11, November, 1959.

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Super. Civ. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

concerned principally with the some 116 watersheds of this range which empt onto the Coastal Plain. These watersheds range in size from 0.1 to 211 square miles.

The rapid increase in the number of debris basins and dams required to provide protection to this expanding populace from debris-laden flood flows has resulted in the impounding and subsequent removal and disposal of the bulk of the debris that debouches from the watersheds. There are presently 45 debris basins and debris dams along the San Gabriel front, and this number will be increased to 56 by 1970. The over-all flood control system also includes 17 flood-regulating dams which are continuously losing valuable storage capacity to impounded debris. Some of these have already been reduced in capacity to such an extent that mechanical debris removal has been undertaken. In a study(1) recently completed, it was estimated that over 300 million cubic yards of debris could be produced from the frontal watersheds of the San Gabriel range over a fifty-year period of storm activity if all storms with a frequency of once-a-year to once-in-fifty-years occurred during the period. This, of course, is an extreme assumption but may be tempered by the occurrence of, say, a thousand-year storm during the period. The situation is made more critical from a financial standpoint in that the cost of handling each cubic yard of debris has risen from $0.33 in 1938 to $1.50 per cubic yard today, and may approach $3.00 per cubic yard by 1970. These costs, when applied to the estimated debris potential stated above, produce dollar amounts in excess of the financial capability of the Flood Control District itself.

Another important aspect of channel treatment is the development and conservation of the water supply that is available from the precipitation that falls on these watersheds. The value of ground water in this area has already reached the vicinity of $20.00 per acre-foot and is expected to approach $45.00 within the next ten years. It should be emphasized that over 50 percen

LOS ANGELES COASTAL PLAIN AND MOUNTAIN PERIMETER
Location Map
Fig. 1

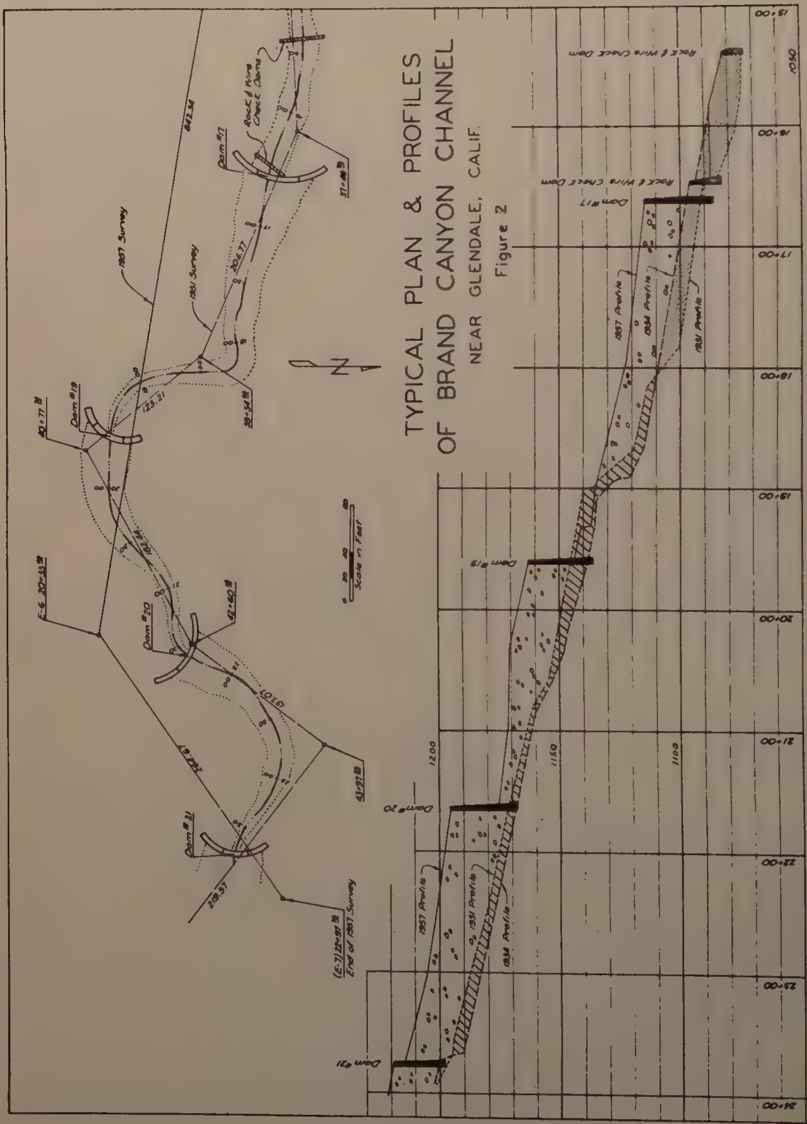
of the water used in the Coastal Plain is still obtained from local sources, primarily ground water.

In order to explore and develop ways and means of reducing the volume of debris being produced from these watersheds, several important treatment projects utilizing check dam systems have been undertaken. Treatment efforts have been based on the theory that if the channels of a watershed can be stabilized, further downcutting into the mountain mass will be halted, and the major source of erosion will have been removed. Bank sloughing, which follows the creation of unstable side slopes due either to downcutting of the channel bottom or to undercutting of the channel banks, is stopped. The importance of preventing bank sloughing should be stressed, for once it starts, it gradually works farther and farther up the hillside contributing large quantities of debris as it progresses, until a stable slope has again been established. In many cases, the erosion scar may continue all the way to the ridge line.

Channel stabilization is also an effective means of halting erosion from previously-created slide areas. By providing a stable toe at an elevated position upon which the material raveling from above may accumulate, and by reducing the erosive power of the stream to wash away this material healing of these areas may be accomplished. A reduction in stream energy occurs as the result of a 25 to 30 percent decrease in channel gradient caused by the deposition on a milder slope of material behind the stabilization structures. Further energy and erosive power is lost through the spreading of the flow over the widened channel. Once the eroded material has begun to build up at the base of the slide area, vegetation may re-establish itself to further aid the healing process.

Other benefits of channel stabilization are realized through the new debris and water storage capacity provided by the system. The pore storage in the debris wedge behind each dam is in essence a small reservoir which affects water conservation in two ways. First, water is stored at time of high runoff and released slowly at times of low flow, thus correspondingly reducing waste to the ocean at times of peak discharge. Second, the saturated debris wedge greatly increases the area of water contact in the canyon bottom thereby increasing deep percolation opportunity. Though the salvage in each of these reservoirs is minute in itself, the aggregate behind a system of check dams in a given canyon is significant. This is a particularly important item in the San Gabriel Mountains because the ground water storage available in the cracks and crevices of this highly-fractured range is far greater than the volume that can be supplied under normal channel conditions.

The first stabilization project of importance was constructed in Brand Canyon, near Glendale, in 1938 (see Fig. 1). This project has proven very valuable in evaluating benefits from this type of treatment. Nineteen dams, ranging in height from 10 to 40 feet above the stream bed, were constructed to stabilize 4,800 feet of channel. Since construction, this system has stored over 80,000 cubic yards of debris. The impounded debris has reduced the average channel gradient through the stabilized reach from an average of 12.5 percent to 7.3 percent, and has elevated the stream bed from its intrenched position an average of 11.4 feet through the reach. A typical reach of the Brand Canyon Channel is shown in Fig. 2 along with the plot of three profiles of the channel bottom. The profiles were taken in 1931, 1934, and 1957. The downcutting of the channel upstream from Station 18+10 between 1931 and 1934 is clearly shown by the respective profiles. In the reach below Station 18+10, however, the stabilizing influence of two small rock and wire



TYPICAL PLAN & PROFILES OF BRAND CANYON CHANNEL NEAR GLENDALE, CALIF.

Figure 2

check dams constructed in 1931 is evident. The 1957 profile shows the aggradation of the channel resulting from the installation of the large check dams.

It is believed that this system has now reached its maximum storage capacity. Therefore, a study(2) of debris production for the 1957-58 storm season was made to measure the effectiveness of the system. By coincidence, this proved to be an excellent season to make the study, in that precipitation was nearly 165 percent of normal. Sunset Canyon, which is an adjacent watershed but with no stabilization treatment, was used for comparison. The study was greatly facilitated by the fact that both watersheds have debris-retaining basins to enable measurement of debris production. Also, physiographic features such as elevation, geology, and compass orientation of both watersheds are very nearly the same. Rain gages were located in both watersheds to reflect any significant difference in precipitation amounts. The drainage area above Sunset Debris Dam is 0.44 square miles, and that above Brand Debris Basin is 1.03 square miles.

The results of this study show that a rate of 8,410 cubic yards per square mile were eroded from Sunset, while the rate from Brand was only 50 cubic yards per square mile. An analysis of the debris produced from each of the watersheds indicated that, for Brand, the maximum particle size was 3/8" in diameter; in Sunset, in addition to the finer material, there was a high percentage of larger material ranging in size up to 1.5 feet in diameter. A study was also made of erosion source areas in and adjacent to the main channels of each of the two watersheds. The results showed that in the 3,570 feet of channel mapped in Brand, 1,850 square feet of active erosion area existed. In Sunset, where 3,030 feet of channel were mapped, there were 5,700 square feet of active erosion area. These two studies indicate that erosion in Brand was limited to sheet erosion; in Sunset, in addition to expected sheet erosion, the main source of debris was the channel and bank areas. Thus, it is seen that through the 1957-58 season the check dam system performed very effectively. The degree of stabilization that has been achieved is shown in Fig. 3 by two comparative photographs of the channel and adjacent side slopes in the vicinity of Dam 19 (see Fig. 2). Photo (a) was taken in 1939(3) and Photo (b) was taken in 1957.

The next treatment project of interest was completed in 1948 in the Arroyo Seco watershed, near Pasadena. This project was also experimental in nature


(a)


(b)

Fig. 3. Dam 19, Brand Canyon, near Glendale, California (Both photos taken from approximate same point, looking upstream).

in that many different types and sizes of structures were employed. The largest of these was the Brown's Mountain Barrier which rises 58 feet above stream bed and has a storage capacity of 1,200,000 cubic yards. A total of 167 structures were installed using five types of construction materials: reinforced concrete, concrete crib, soil cement, steel bin, and steel arch. Accurate cost records were maintained on all phases of the project to enable an evaluation of the cost of each type. Although this project provided much valuable data relative to structure size, design, and desirable construction materials, it proved to be very costly. As a result, a damper was placed on upstream engineering in this area for a number of years.

By 1956, for reasons previously stated, it became evident that upstream engineering was an essential phase of the over-all flood control effort. It was therefore decided that a new project should be undertaken that would incorporate the experience gained from previous projects in this area, as well as that available from similar works in Europe and elsewhere. Following a thorough study, it was decided that the design and construction of this project should be based on the following general criteria: (1) the spacing of structures should be such that the impounded debris behind each extends to the toe of the next structure upstream, (2) in general, concrete crib-type construction should be used, (3) the spillway height should not be less than 10 feet above the stream bed nor greater than 17 feet, (4) a stabilized debris slope of 0.7 of natural gradient should be used for the spacing of structures, (5) the structures should be designed to meet all requirements of a gravity-type overflow dam, (6) stabilization treatment should be limited to channel gradients of 20 percent and less, and (7) production and assembly line methods of construction should be employed wherever possible.

Cooks Canyon, near La Crescenta, was selected as the watershed to be treated. The project was planned and constructed cooperatively by the Flood Control District and the U. S. Forest Service and was completed in late 1956. An analysis of project costs(4) indicated that the channel had been stabilized at a cost of approximately $40 per foot through a reach of channel that varied in gradient from 6 percent in the lower portion to 10 percent in the upper. This is a considerable reduction in cost over previous projects.

Two other projects have since been started, using the same basic design criteria with a few minor modifications. These projects are in Monrovia and Santa Anita Canyons, above the cities of Monrovia and Arcadia, respectively.

The Santa Anita Project(5) is of particular interest because it is the first attempt to stabilize all main channel reaches of a somewhat larger watershed (11 square miles). This project, when completed, will consist of 70 check dams and will stabilize 50,000 feet of channel. The cost of this project is estimated to be $1,750,000. Fig. 4 shows the over-all stabilization plan. Fig. 5 shows Dam 4 which is typical of the concrete crib structures of this project.

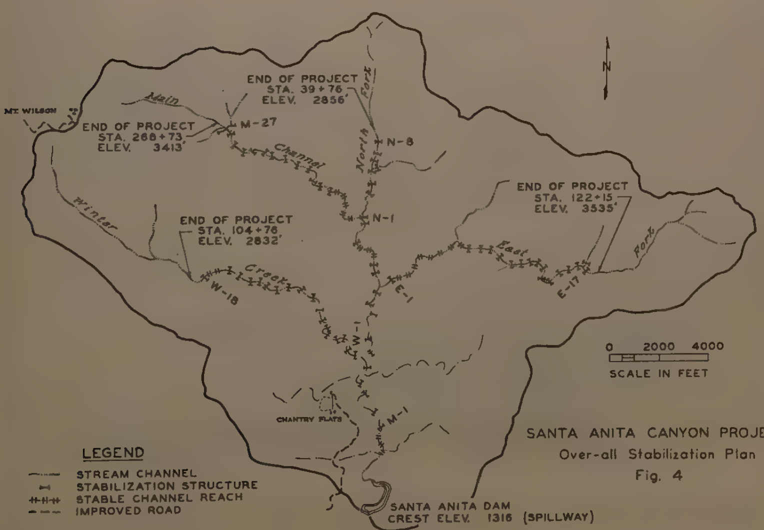
The system will provide over 757,000 cubic yards of debris storage capacity within the watershed. This capacity is of special importance because the Big Santa Anita Dam, located at the mouth of the canyon, has already lost 56 percent of its capacity through siltation which has seriously impaired its usefulness for flood-regulation and water conservation. As an example of the debris movements that take place in the San Gabriel frontal watersheds when subjected to a fire-flood sequence, two relatively minor storms that occurred in the Santa Anita watershed in January 1954, following its burning over between Christmas and New Year's, produced 208,100 cubic

yards of debris with a peak discharge of only 1,240 c.f.s. attributable to the larger of the two storms. During the 30 years since the reservoir was completed, debris has entered at the average rate of 74,200 cubic yards per year. Assuming this rate to continue, the storage capacity provided by the project is ample to hold all debris eroded within the watershed for more than 8 years. Through sluicing finer materials reaching the reservoir into the downstream channels, it would then be possible to reduce storage loss to about 35 percent of the original capacity. It is also anticipated that the debris production from the watershed will be greatly reduced and that any debris that does enter the reservoir will be fine material which can more readily be sluiced.

Water conservation benefits, as previously discussed, will be realized from the 164 acre-feet of water storage capacity that will be available in the impounded debris. This volume is based on an estimated debris porosity of 35 percent. The pore space in the 70 small reservoirs will fill during times of storm runoff and drain out slowly as the storm subsides. This cycle may repeat itself several times during a single season, depending on storm spacing and duration. Therefore, a somewhat larger continuous supply will be available to the spreading grounds in the valley below and waste to the ocean at times of sizeable flood runoff will be correspondingly reduced. The increase in area of contact for deep percolation will be particularly great in this watershed due to the broad high-water channel that will be made available for percolation.

An excellent opportunity is available to study the effect of treatment on water yield, peak runoff, and base flow inasmuch as continuous rainfall and runoff records have been collected for this watershed for a period of over 30 years.

The conservation of water is also the objective of another important channel treatment project that is currently being put into operation in Monroe Canyon, near Glendora. This project, which has been undertaken by the



SANTA ANITA CANYON PROJECT
Over-all Stabilization Plan
Fig. 4

California Forest and Range Experiment Station of the U. S. Forest Service, is a study to test on whole watersheds the promising and practical methods of increasing water yield by the control of riparian growth. The study will also determine how increases in water yield differ with variations in rainfall, soil, geology, topography, and vegetation. The influence of these factors must be determined because only with these basic answers can this type of treatment have widespread application to other watersheds. Other agencies of government are also cooperating in this project to determine its effects on quality of water, on erosion, and on peak runoff. The Monroe watershed and adjacent Volfe watershed, which is to serve as a control, are located within the San Dimas Experimental Forst, and both have been carefully calibrated in their natural state for the past 20 years. During the past summer work was begun on the removal of all woody growth from the canyon bottom and lower side slopes. The vegetation removed consisted of an overstory of large trees such as live oak and alder and an understory of brush. The treatment, when completed, will involve about 80 acres of the 875-acre watershed.

It is anticipated that native grasses will occupy the treated area; however, chemical controls of woody regrowth will be required. The clearing program also includes the removal of all floatable debris from the channel area to prevent temporary damming and bulking of storm runoff. It is too early in the study to give valid costs or specific estimates on effects of treatment. It is understood that such results will be released by the Experiment Station as they become available.

The channel treatment projects in the Arroyo Seco, Cooks, Monrovia, and Santa Anita Canyons were undertaken jointly by the Los Angeles County Flood Control District and the U. S. Forest Service as part of the Los Angeles River Watershed Flood Prevention Program. It should also be noted that

Fig. 5. Dam 4, Santa Anita Canyon, near Arcadia, California (Looking Upstream).

much has been accomplished in regard to fire improvements, vegetative cover improvement, and road slope stabilization as a part of the over-all watershed treatment programs.

SUMMARY

This paper has presented the background and some of the details of a program of research and developments aimed at reducing the volume of debris that must be handled by the Flood Control District. The stabilization of the main channels of a watershed by mechanical means has emerged as the most practical method of accomplishing debris reduction. Mechanical stabilization by the use of check dams reduces erosion by halting channel downcutting and by reducing the occurrence of bank sloughing. Other benefits also provided are the new debris storage capacity behind each of the check dams and the conservation of water through direct storage and increased percolation opportunity. The Big Santa Anita Project, near Arcadia, which is presently under construction, represents the first full-scale use of this type of treatment in this area. The project, when completed, will stabilize 50,000 feet of main channel in this 11 square mile watershed. The project cost is estimated to be $1,750,000.

ACKNOWLEDGMENTS

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ELECTRONIC COMPUTERS USED FOR HYDROLOGIC PROBLEMS

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SUMMARY

The need for solving complex hydrologic problems rapidly has led Bureau of Reclamation engineers to the utilization of electronic computers for such studies. Applications have been made to problems in water resources, water requirements, water utilization, flood hydrology, and sedimentation. Optimum solutions of high accuracy are obtained quickly and economically.

INTRODUCTION

The need for the rapid solution of a variety of complex hydrologic problems has led Bureau of Reclamation engineers to the utilization of electronic data-processing equipment. Hydrologic problems encountered in the planning, design, construction and operation of multipurpose water resource projects are, in many instances, ideally suited for the use of these machines since such problems often require the solution of complex mathematical relationships, the reduction of large volumes of data, the frequent repetition of a basic operation, or the evaluation of alternative assumptions and criteria.

In past years, Bureau engineers have obtained sound solutions to many of their hydrologic problems by short-cut, empirical methods. Continuous hydrologic research has, however, resulted in the development of many complex mathematical relationships, and the operation of expanding hydrologic networks has resulted in the accumulation of large volumes of basic data. It would, in many instances, be impracticable to utilize fully these relationships and data by the usual manual methods because of the greatly increased costs to the water and power users and other beneficiaries. Fortunately, the development during recent years of electronic data-processing equipment has

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offered the engineer an opportunity to utilize the new procedures and data without greatly increasing his requirements for technical assistance.

The benefits of utilizing the new equipment may range from the simple testing of tried and proven empirical relationships, in light of new technical knowledge and data, to the actual solving of involved mathematical relationships. The conversion of evaporation pan data to obtain gross reservoir evaporation is an example of the former benefit, while the determination of total sediment transport by the Modified Einstein Procedure, or the analysis of large volumes of data by multiple correlation procedures, are examples of the latter. Whenever an empirical relationship is tested and found valid, it is retained and improved as may be indicated. The complex equations and masses of data are utilized as economically acceptable methods of solution are developed. Herein lies the challenge to the engineer in application of electronic data-processing equipment.

Initial Considerations

The application of electronic calculating equipment to hydrologic problems in general and the introduction of machine computing procedures into an organization require a detailed knowledge of equipment capabilities and limitations on the one hand and a full knowledge of the hydrologic requirements of specific problems on the other hand. With hydrology studies, as with all problems for which electronic computers are used, it is essential that the problems be completely and accurately defined before any attempt is made to utilize such equipment in their solutions. Even though a problem is fully defined, it may be a somewhat difficult procedure to program the desired solution on an electronic computer. Without such definition, it usually will be impossible to program any solution.

The engineer who has hydrologic problems to solve must first know his problems. However, he also must know the primary capabilities and limitations of computing equipment so that he can determine whether or not his potential applications are indeed proper problems to put on such machines. Full integration of problem know-what and machine know-how is essential to the efficient and effective adaptation of studies to machine solution. A proper decision to automate can result in optimum solutions being obtained quickly and economically. An improper decision can result in added confusion and costs.

Scope of Hydrologic Analyses

It is the desire of the Bureau's engineers to make trial applications of electronic data-processing equipment to sample problems covering the entire scope of hydrologic problems involved in the planning, design, construction, and operation of Reclamation projects. It is believed that this procedure will provide a basis of judgment for further and more detailed utilization of the equipment. Such problems may be divided, roughly, into the categories of water resources, water requirements, water utilization, flood hydrology, and sedimentation. Within these categories, some of the problems which appear to offer the greatest possibilities for machine analysis and which are of particular interest to the Bureau of Reclamation are:

'ater Resources

To extend basic records of streamflow, precipitation, and other similar ictors and to fill gaps in those records.

To evaluate changes in observed hydrologic data in relation to climatic nd physiographic changes, changes in record keepers and equipment, and epletions resulting from man-created developments.

To adjust historic observed hydrologic data to constant levels of develop-ient.

To analyze land use and watershed treatment data in relation to stream-ow, precipitation and other hydrologic data so as to determine whether or ot changes in streamflow can be attributed to land treatment.

To forecast seasonal runoff by use of multiple correlations of meteoro-gic and physiographic data.

'ater Requirements

To correlate climatic, physiographic, and other pertinent data with records f water deliveries for irrigation.

To analyze data pertaining to domestic, municipal, and industrial water equirements in terms of related factors such as location, climate, and popu-ition.

'ater Utilization

To conduct coordinated operation studies of multiple-purpose water re-ource developments that involve various combinations of water resources, ater requirements, and facilities.

lood Hydrology

To evaluate flood magnitudes and frequencies in terms of observed hy-rologic, meteorologic, and physiographic data for specific streams or areas.

To route floods through streams, lakes, and reservoirs or combinations iereof.

To forecast flood runoff using meteorologic, hydrologic and physiographic ata.

edimentation

To derive flow-duration curves and sediment-rating curves from adjusted reamflow and sediment sampling data.

To estimate sediment-yield rates by correlation of meteorologic, hydro-gic, physiographic, and sediment data.

To calculate total sediment load transported by a stream.

Sample Problems

Trial solutions by electronic data-processing equipment have been com-eted on a few of the problems mentioned above and are described in the llowing paragraphs.

Extension of Records

Extensions of streamflow records for the proposed Waurika and Arbuckle Projects in Oklahoma were necessary for estimating water supply and storage requirements. Because of the very short periods of record available on the two streams involved, it was necessary to develop rainfall-runoff relation ships in neighboring areas of similar drainage characteristics. These relationships were explored by machine methods, by developing a multiple regression of monthly streamflow in relation to concurrent precipitation and antecedent moisture. Streamflow records which were utilized are:

Arbuckle Project	
Rock Creek near Dougherty	March 1956 to September 1957
Waurika Project	
Beaver Creek near Waurika	June 1953 to September 1957
Neighboring areas	
Little River near Tecumseh	October 1943 to September 1957
Blue River near Blue	June 1936 to September 1956
Little Wichita River near Archer City	March 1941 to December 1955

The analysis involved a total of 282 trial correlations from which the more consistent and reliable estimating equations were selected. Equations were developed first for Little River, Blue River, and Little Wichita River from monthly and seasonal correlations of precipitation and runoff. These equations were applied to the Rock Creek drainage by substituting Rock Creek precipitation data and adjusting for area of drainage basin. The resulting estimates of streamflow were compared with available records of Rock Creek at Dougherty in selecting the most suitable equation for extending the Rock Creek records. Although the streamflow records on Beaver Creek, Waurika Project, appeared to be adequate for the derivation of a precipitation runoff equation for extending streamflow records, the equations for the other basins mentioned above were used as a check on the Beaver Creek estimating equation.

Multiple-correlation computations for the 282 equations involved in these studies were made on an electronic computer in about 7 machine hours. At $80 per hour, the machine rental was $560. Labor charges have not yet been fully defined but will be about $2,500. A direct comparison with a manual solution is not available, but previous experience indicates that, for equivalen detail, manual solutions would have been much more expensive. Actually, of course, had the electronic computer not been available, the detail and resultant accuracy would have been substantially reduced.

Land Use and Watershed Treatment in Relation to Streamflow

An extensive study of the effect of land use and watershed treatment on streamflow is being conducted cooperatively by the Agricultural Research Service and Soil Conservation Service of the U. S. Department of Agriculture and the Bureau of Reclamation of the U. S. Department of the Interior. It is hoped that one of the products of this study will be a reliable expression for determining runoff as a function of related factors. Since there are many factors which influence runoff from a watershed, multiple correlation provide a means for obtaining such an expression or estimating equation.

The study group has examined precipitation and runoff records for many reas ranging in size from small experimental plots to large drainage basins. xploratory analyses have included various combinations of precipitation, oisture indices, and other variable on an annual, seasonal, monthly, and dividual storm basis in an effort to develop rainfall-runoff relationships nder changing levels of land use and watershed treatment.

This exhaustive study is continuing. Electronic computers have permitted camination of combinations of variable which would not otherwise have been ossible. Manual processing would have been prohibitive due to the expendi- ıre of both the time and money required.

orecasting of Seasonal Streamflow

Runoff during the snowmelt season, which is of major concern in project anning and operation, is influenced by several factors on which data are vailable. Several of the principal factors are water content of snow antece- ent to the runoff season, previous precipitation (as an index of watershed onditions), and precipitation during the early part of the runoff period. Each these factors involves numerous measurements which require investigation the development of a forecast equation.

As described by Ford,[1] "the problem is to develop a multiple-regression quation which summarizes the relationships of past hydrologic events as videnced in records of natural runoff and of factors contributing to runoff, nd to determine the reliability of this equation as a means of forecasting." past years the Bureau of Reclamation has developed numerous forecast quations by application of multiple-correlation analysis to many river basins the western states. Many of these computations have been accomplished anually using desk type calculating machines.

Any exhaustive study of forecasting possibilities would involve large nounts of computational work—prohibitive amounts so far as manual pro- essing is concerned. Extensive machine processing of data for the develop- ent of forecasting procedures for the operation of a Bureau of Reclamation ultipurpose reservoir system in northern Colorado and southeastern yoming was conducted over a period of 3 years and involved more than 500 ial equations. This study has been described by Koelzer and Ford.[2] Con- stent improvements in forecasts were made during the period of the machine nalyses.

oordinated Reservoir Operation Studies

Coordinated operation studies of systems of multiple-purpose reservoirs nd powerplants pose one of the more intricate and time-consuming hydro- gic problems solved by engineers of the Bureau of Reclamation. These udies may be prepared to serve a variety of purposes, such as: determina- on of reservoir, powerplant, outlet and conveyance system capacities; ef- ctive regulation of streamflow to provide dependable releases for irrigation, unicipal water, prior rights, hydroelectric power generation and other water ses; flood control; or economic analysis. A typical study involves the utili- tion of historical, depleted, or adjusted streamflows at specific points thin a river basin. The flows are routed through a system of reservoirs nd powerplants into a conveyance system and to points of use such as blocks irrigated land or municipalities. Losses due to evaporation, seepage, nsumptive use, accretions due to local inflows, return flow, and bank

storage are taken into account as they occur in the process. The studies are usually prepared to cover an entire period of record of perhaps 25 to 50 yea paying particular attention to periods of critical droughts or maximum flood when full reservoir capacities will be utilized.

The electronic data-processing machine is a rapid and efficient device fo processing a series of operation studies. It may be expensive to plan and program a complex operation study, and a number of studies usually must be processed to realize a saving over manual computation methods. The studie usually evolve into the use of a number of mathematical expressions with ap- propriate tests being incorporated in the computer program to permit the choice of the proper relationships to fit the given problem conditions. Flexi- bility must be incorporated into the program so that all of the desired solu- tions can be obtained by changing only certain independent variables or con- trolling criteria. The precise planning, programming, testing, and coding of the operation studies may be tedious, time consuming, and expensive, but savings in time and money can be realized for a series of studies.

Recently, the necessity for computing a series of operation studies on the Upper Missouri River offered an opportunity to utilize electronic data- processing equipment. In this instance, it was desired to operate the Bureau Canyon Ferry Reservoir and Powerplant in coordination with or without othe powerplants to determine the hydroelectric power production for various con ditions. More than 4,500 individual entries were needed to compile the re- quired basic data. These data included the runoff records of the Missouri River and its tributaries from Townsend to Great Falls, Montana; potential river depletions; evaporation rates; reservoir area-capacity tables; power and pumping plant characteristics, such as efficiency, capability, and capacit runoff forecast equations; and flood-control limitations. About 3,500 locatio of magnetic drum storage were utilized to store the program and data, or, in other words, 3,500 separate computer instructions and values of initial input data were required to direct and compute the study.

Calculation of Total Sediment Load

With the development of the Modified Einstein Procedure[3] and publica- tion of a step method for computing total sediment load by this procedure,[4] the Bureau of Reclamation initiated a total-load sampling program on vari- ous streams where sedimentation is a critical problem in the planning, desig and operation of Reclamation projects. The total-load sampling programs o the Middle Rio Grande and Lower Colorado River were initiated to aid in comprehensive channelization plans and designs. On the Lower Colorado River between Davis and Imperial Dams, 18, 88, 102, and 104 total load samplings were made during 1955, 1956, 1957, and 1958, respectively. Since the Modified Einstein Procedure requires long and involved calculations, the decision was made to investigate the possibilities of making the calculations on an electronic computer. In connection with Missouri River problems, the Division Engineer of the Corps of Engineers at Omaha, Nebraska, had pro- grammed a total suspended load procedure on an electronic computer. The total suspended load calculation is a part of the total load calculation by the Modified Einstein Procedure, and similar functions are used in both calcula- tions. In collaboration with Corps of Engineers' personnel, a computer flow chart of the Modified Procedure was developed. The program has been com pletely coded and checked out with sample problems solved previously by manual methods.

The program was developed primarily for the computation of total loads in e Lower Colorado River. It is, however, a general program which can be ed to calculate total loads for any stream. The possible ranges in magni-des of input data and output results, together with foreseeable changes which ght be made in the basic computation procedures, were all studied prior to ogramming, so that basic data from almost any stream could be used to de-rmine total sediment load.

It appears that the development of the computer program for the Modified nstein Procedure will result in substantial benefits. Assuming 100 total-ad sampling measurements per year, the amount of time and money spent nually on manual calculations, including checking, would be about 75 man-ys and $1,900, respectively. Initial costs of program development will be out $2,500 but, after that, annual costs for the computer processing of 100 ts of sampling data will be about one computer-day at a cost of approxi-ately $700.

Besides realizing tangible benefits, the developed program can be used with xibility for research into modifications of the Procedure. The Modified ocedure is not completely satisfactory at this time, and various parts of e initial program can be changed easily to investigate modifications and eir effect on the computed total load.

en Canyon Reservoir IBM Sediment Inflow Study

To provide the basis for a reliable estimate of sediment inflow to Glen nyon Reservoir, which will be formed by Glen Canyon Dam now being built the Colorado River near the Utah-Arizona line, a flow-duration sediment-ting curve analysis was initiated in January 1957. Data from the Geological rvey stations at Grand Canyon and Lees Ferry were used in the study. The es Ferry station measures the approximate contribution of the main lorado River to Glen Canyon while the Grand Canyon station measures the ntribution of the Paria and Little Colorado River in addition to the main lorado River.

Some preliminary studies were conducted to determine seasonal breaks the sediment rating curve and period breaks in runoff characteristics. om these studies, the yearly runoff variation was divided into two parts each station. The sediment variation was divided into four parts for each tion.

The study was performed at a cost of approximately $1,500. If the total dy had been done by manual computation, the cost would easily have ex-eded $5,000. One important benefit of this study is that the complete runoff l sediment records through 1955 are available for any other type of study ich would utilize part or all of the station records. With the addition of a years' records, the study can be kept completely up to date for a small st. Variations of the study are possible, using different periods or season-breakdowns, with little effort or cost. Manual computation of study varia-ns would require almost as much time and expense as an original manual dy.

This study was simple in comparison to some detailed programs on elec-nic computers, but resulted in an analysis of approximately 50 years of tion records (24,516 daily water discharges and 13,772 daily suspended liment loads) for a moderate cost and in a comparatively short period of ie.

Approach to Computer Solutions

When hydrologic problems are solved manually using a desk-type calculators, blank forms are often prepared to provide a guide for the computation to be made and to summarize the individual numerical operations. As values are required for measured data, trigonometric functions, logarithms, powers of numbers, or quantities determined from particular mathematical relationships, they are obtained by the engineer by searching tables, reading charts, or scanning tabulations of experimental or computed data. The order of making the computations and the procedures used are usually so chosen that the calculations will proceed in a manner as efficient as possible.

When electronic computing equipment is utilized in obtaining solutions for hydrologic problems, the method of attack is often considerably different than the manual approach. The order in which the calculations are made is so chosen as to take full advantage of the inherent logical capabilities of the stored-program computer; such as the ability (a) to choose the proper computational steps to be carried out for a particular problem through the use of "branching" operations; (b) to modify instructions according to a prechosen plan and the conditions of the problem; and (c) to perform repeated calculations of a given type through "looping" operations and subroutines.

It is often more efficient to have the computer calculate the value of a trigonometric function each time it is needed than to store a large number of tabular values of the function in the memory of the computer. Such an approach would not normally be followed, of course, in any manual procedure. Experimental or measured data may often be represented to the desired degree of accuracy by a mathematical expression obtained to "curve-fitting" methods. Here again, the computer would be given instructions to calculate a value of the expression each time it is required. When tables of data are such that they cannot be expressed properly or adequately by a mathematical relationship, it is often possible to have the computer determine appropriate values to be used in a given calculation through the use of "table look-up" commands or routines. When a study requires the use of a mathematical relationship which is difficult or impossible to express in elementary functions approximate expressions(5) may often be employed with a computer to give the desired results. New techniques and procedures in numerical analysis are constantly being developed for use with electronic computers which permit the application of these machines in many areas of study hitherto considered impractical or impossible.

Many programming aids are now available which help to reduce the amount of time and effort required in the programming and coding of problems for computers. Through the use of assembly and compiler routines, it is possible to have the computer itself assist in the preparation of a program for a given problem. Such procedures illustrate further the fact that the approach to the solution of a problem using an electronic computer may be considerab different from that used with manual methods.

SUMMARY

The use of electronic data-processing equipment on hydrologic problems, such as those noted in this paper, has been studied to determine the associated advantages and disadvantages together with the capabilities and

mitations of the machines. It is apparent at once that, while the electronic omputer offers many benefits and advantages, it is not a substitute for sound ngineering analysis, experience, or judgment. The most obvious reasons for ilizing such equipment are to reduce the time and cost required for calculaons; to provide more data upon which more intelligent decisions can be ased; and to free the engineer from the drudgery of routine manual computaons and permit him to do more important professional work. That savings time and money can be substantial has been demonstrated in previous disussions of individual problems.

From a purely technical viewpoint, one of the major advantages of maine solution lies in the ability to obtain information beyond the economic mitations of manual processes. For a given problem, a variety of criteria nd basic assumptions can be examined with relatively simple adjustments in e program.

Use of the electronic computer also permits a more detailed solution hich usually results in a higher degree of accuracy in the end product.

Perhaps the most important advantage of electronic computation from the iewpoint of the planner is the ability to compare the effects of alternative chemes of development. Reservoirs, powerplants, conveyance structures nd other physical works may be left in or taken out, and capacities may be hanged with only slight modifications in the computer program once it has een worked out correctly. The machine rerun time is usually a modest item. hanges in criteria for water storage and release and for power generation ay be made and completely rerun with little increase in cost. The addition r deletion of major project functions such as irrigation, power, or flood ontrol usually requires a basic change in program and, therefore, may reilt in substantial increases in cost. Once a complete program has been preared, say for a multipurpose hydrologic system, alternate schemes of operaon may be investigated thoroughly, quickly, and economically. The end esult is the ability to determine the optimum multipurpose operation solution ather than merely what might be a satisfactory solution obtained by manual ethods. More intelligent decisions can thus be made because more informaon is available to support those decisions.

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DIGITAL COMPUTERS FOR WATER RESOURCES INVESTIGATIONS[a]

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ABSTRACT

Digital computing equipment is now being used by the U. S. Geological Survey to analyze published streamflow data and to process data obtained in connection with water-loss studies. Basic streamflow data are to be recorded in the field on punched paper tape.

SUMMARY

Digital computing equipment is now being used by the Geological Survey to analyze published streamflow data and to process data obtained in connection with water-loss studies. Field recording of basic streamflow data in digital form on paper tape is about to begin, and a general-purpose digital computer will be used for the computing and tabulating. Statistical analyses of streamflow data are now being made that hitherto were considered impractical because of the tremendous mass of data involved. Water-loss data are being processed rapidly and the results are available much sooner than if the data were handled manually. Trained technical personnel are being relieved as much as possible from monotonous chart processing.

INTRODUCTION

The possible use of modern digital computing equipment in hydrologic studies has been under investigation by the Geological Survey since 1950. The computation of daily discharge at about 7,000 gaging stations is a

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time-consuming task, and one that appears well suited to mechanization. Statistical analyses are needed to make best use of streamflow data. Many such analyses are not made simply because of the tremendous mass of data involved.

Many different meteorological and limnological data are obtained in water-loss studies. Manual data processing is slow and laborious. The computations are of a type that could be made efficiently using a digital computer. Studies by the Geological Survey have been directed principally toward the use of modern digital computing equipment for processing streamflow records and analyzing the results, and processing field data used in water-loss studies.

The Geological Survey is currently making full-time use of its general purpose digital computer, a Datatron 205, in Washington, with about one-fourth of the available time being used for hydrologic studies. Other studies are being made at Denver, utilizing a similar computer at Denver Research Institute on a part-time basis.

The work done on these computers has been so successful that plans have been made to expand the capacity of the Washington facility by replacing that computer with one having an internal operating speed 10 times faster.

Use of digital computers for streamflow computations and analyses

Before any decision could be reached as to whether it would be economical for the Geological Survey to obtain a general-purpose digital computer, preliminary investigations were made to determine the type of studies that might be advantageously made with the computer. The ease with which extensive statistical analyses of streamflow records might be done on such a computer was considered. The Geological Survey has been collecting records of streamflow for many years, but few analyses of the data had ever been made. In order to solve certain problems relating to streamflow, a user of our records first had to abstract from published yearly reports hundreds or perhaps thousands of figures of daily discharge for a given stream-gaging station and then arrange and summarize them in the manner required for his analysis of the records. Several users of the records might duplicate each other's analysis almost exactly. The availability of a general-purpose digital computer made possible for the first time the preparation of standard statistical analyses on a large scale which could be used as the basis for the majority of individual studies involving figures of daily discharge. Three basic analyses of daily stream discharges were selected as follows:

1) A magnitude-frequency analysis of the daily discharges as individual items (duration table) each water year (Oct. 1 to Sept. 30);

2) the lowest average flows for certain numbers of consecutive days within each low flow climatic year (Apr. 1 to Mar. 31); and

3) the highest average flows for certain numbers of consecutive days within each water year (Oct. 1 to Sept. 30).

These computations are relatively simple for a digital computer. For instance, duration tables are computed using a series of operations very much like those performed when the computations are done manually. That is, frequency distribution class limits are chosen for each station and the number of daily discharges in each class is tallied. The totals in each class

for each year are printed. Similar totals for the entire period of record are also printed. These totals are then used to compute the percent of time the flow equaled or exceeded each of the class limits.

The computation of lowest flow for certain numbers of consecutive days of flow is even simpler. A running sum is computed for the number of days under consideration. Each time a sum is obtained it is compared with the lowest sum for that number of days previously obtained. If the new sum is less than the previous lowest sum, the new value is substituted. Then the first value of the series of discharges is subtracted, the next discharge ahead of the old series is added, and the comparison with the previous low sum is again made. When the end of the year is reached the lowest sum thus obtained is divided by the number of days to obtain the lowest average discharge for that number of consecutive days. Next a new number of days is substituted for the old number and the whole process is repeated. After the results for all the required numbers of consecutive days are obtained, the results for that year are read out of the computer. High-flow summaries are computed in a similar manner.

It is obvious that the computation is simple, but the number of data is large. The biggest problems encountered were finding an efficient method of transcribing the masses of data into a form suitable for direct computer input and developing procedures for verifying the prepared data. For instance, after a detailed study of various possible storage media, paper tape was chosen. The most important reason was that the data consist primarily of one value (discharge) for each time period (a day) and the data can always be used in rigorous time sequence. Therefore, mechanical sorting is not needed, and advantage can be taken of the smaller amount of space required by a unit of data on paper tape than on cards. Also the ability of the Datatron 205 to read paper tape photoelectrically at high speed (540 characters per second) made the computer handling of paper tape rapid and efficient. Moreover, relatively inexpensive portable machines could be used to punch paper tape, thus permitting much of the data transcription to be done in the field offices where the records were collected rather than in Washington. Add-punch machines have been used for most of the paper tape preparation to date. These machines add and punch simultaneously, which allows using previously computed totals on the source document as a check. However, data for computer input must be practically without error if the computer is to be used efficiently, so even this check on previously derived totals was not sufficient. Therefore, an editing program was prepared for the computer to have it detect punching errors so that corrections could be made before doing the actual computations. Although it may appear expensive to use the computer to find errors in the tape, this procedure has proved to be the most practical solution, and the computer is now used to edit all data tapes. About 25 seconds per station-year is needed to make six validity checks. After the machine editing is done, the actual computations, which require about 4-1/2 to 5 minutes per station-year for the three basic analyses, are seldom interrupted because of errors in the data. To date the computer has been used to analyze about 30,000 station-years of record.

Much work has also been done in developing means for using the computer in processing raw streamflow data. Preliminary studies were begun long before the acquisition of a general-purpose computer was considered. This computer application is much more complicated than analysing results already computed, and a solution is only just now at hand.

For years stream stages have been recorded on strip charts. This is an analog record. As processing these charts is laborious and time-consuming, it has long been recognized that the use of automatic data handling equipment would be most desirable. The first development was a special-purpose computer to scan the standard strip charts photoelectrically, determining the gage height at uniform time intervals, converting each gage height to its corresponding figure of discharge, and integrating to obtain the daily total. The basic design of this special-purpose computer was simple, but many unanticipated complications were encountered when the computer was built. It was completed, however, and was operated experimentally for a short time. Although this device could not be made to operate with adequate reliability and accuracy for continuous use, a great deal was learned from it. The most important thing was that special-purpose computers are inherently expensive, because development and testing costs cannot be spread over many similar devices as they can be for general purpose computers built in quantity. Therefore it was decided to utilize general-purpose computers as much as possible.

If the basic data were recorded directly in suitable form, a general-purpose computer could then be used for almost all of the data reduction. But until very recently the available recording instruments were either too expensive or required too much electric power. Cost is important because several thousands of field installations are involved. Standard electric power is not available at most gaging stations. A slow paper-tape punch especially designed to produce punched tape exactly as required by our general-purpose computer was tested, but it also required too much power for battery operation and cost too much. A new device developed within the last year seems to be the solution to the problem. This is a simple mechanical coding device attached to a slow-speed punch which will record a four-digit observation in a single row of holes on a wide paper tape. As originally designed, the punch had a capacity of only three decimal digits, and required 110-volt power, but the manufacturer modified it to record four decimal digits and to use battery power. Fortunately, current drain is extremely low, so that a low-cost dry-cell battery should operate the device for a year.

Although this parallel-coded wide paper tape can be used in one commercial paper-tape reader, it is not suitable for direct input to any of the present general-purpose computers. A special paper-tape translator had to be built to repunch the data contained in the parallel-coded wide tape onto serial-coded tape. Computer instructions are inserted automatically by the translator where necessary. However, in one important respect the translator is a blessing rather than a burden. Each of the present-day computers using paper tape has its own distinctive coding and format for paper tape input. If thousands of gaging stations were equipped with punches which would produce paper tape for one specific computer, newer computers could not be used unless each field instrument were modified, which would be prohibitively expensive. By using the parallel-coded wide tape, the data can be recorded in the least possible space and the translator, with its easily changeable coding matrix and sequencer, can be used to put the data into the particular coding and format required for input to any computer.

This new digital water-stage recorder will involve the following sequence of operations. Stream gage heights will be punched on wide paper tape at the river bank installation at intervals of 1 hour, 1/2 hour, or 1/4 hour. About once a month the tapes will be removed and sent to a local area office for

translation into tapes suitable for direct computer entry. These translated tapes will be sent to the computing center and fed to the computer, each with a manually prepared tape containing the stage-discharge relation for the gaging station. Daily discharges will be computed and tabulated and at the same time stored on magnetic tape for future use. The printed tabulations will be sent back to the field offices which operate the gaging stations so that the originally computed results can be examined for possible discrepancies. After any necessary corrections are indicated, the tabulations will be returned to the computing center so that the information on magnetic tape may be corrected. After this, the computer can be used to print tables of daily discharge in a form suitable for direct offset printing of the annual series of Geological Survey water-supply papers. The data on magnetic tape will then also be available for statistical analyses such as those previously described.

This system of data collection and data reduction is now being put into operation. The prototype of the digital water-stage recorder has been subjected to extensive environmental testing and the production models are coming off the line. The special paper-tape translator has been tested and found satisfactory. The computer programs involved have been written and "debugged." The first 100 field installations will be made this summer. Within a few years we expect to have thousands of such instruments in the field and a full-scale automatic data-collection and data-reduction system in operation over much of the country.

Meanwhile, an intermediate system of data reduction has been developed to permit one of our large field offices to use the computer to do some of the computation of daily discharges that is now done manually. Data taken manually from the strip chart are read into the computer. The output is a complete tabulation of daily discharges and the usual monthly and yearly summaries. Included in this program is a tabulation of discharges at indicated times suitable for plotting flood hydrographs for all periods above a given flood base.

Other types of hydrologic computations have been made in Washington and many more will be made as the capacity of the computation facility is increased. Smaller projects already completed include computation of flow in tidal streams, statistical correlation of physical factors influencing runoff, recomputation of special discharge measurements to evaluate the effects of the number of sections taken, and computation of tables of mathematical functions used in ground-water hydraulics problems.

Use of digital computing equipment in water-loss studies

Since the Lake Hefner evaporation studies of 1950-51, the Geological Survey has continued its research in techniques for measuring evaporation losses from lakes and reservoirs and has investigated methods of determining evapotranspiration losses from vegetation covered surfaces. In both the evaporation and evapotranspiration studies the basic technique has been the energy-budget method, in which an accounting is kept of all incoming and outgoing energy, the residual being the energy utilized for evaporation. The effect of changes in energy storage is taken into account.

Continuous records of many of the items needed for the solution of the energy-budget equation can be obtained at a centrally located meteorological station. These data include the following:

Q_s, the solar radiation incident to the surface

Q_r, the reflected solar radiation

Q_a, the incoming long-wave radiation from the atmosphere

Q_{ar}, the reflected long-wave radiation

Q_{bs}, the long-wave radiation emitted by the body of water or the vegetation covered surface

T_a, the air temperature

T_w, the wet-bulb temperature

For reservoir evaporation studies it is simpler and sufficiently accurate to compute Q_r using an empirical method developed by Anderson.(1) For evapo transpiration studies, however, little is known of the reflectivity of the surface, and it must be measured. For evaporation studies it is simpler to compute Q_{bs}, as it depends only on the emissivity of water, which has been accurately determined, and the water surface temperature, which can be measured with little difficulty. A total hemispherical radiometer can be used to measure the incoming radiation. For evapotranspiration studies, however, both the emissivity of the vegetation covered surface and its temperature are unknown, and it is therefore necessary to use a net exchange radiometer, which measures the difference between incoming and outgoing radiation. The air and wet-bulb temperatures are used in computing the Bowen ratio.

The output of each of these instruments is a potential difference or voltage With the use of suitable scaling devices, all items can be recorded on one multi-channel recording potentiometer. Ordinarily an 8-channel instrument is adequate. With this instrument, the various items may be recorded at 30-second intervals so that with an 8-channel recorder, each item is recorded once in 4 minutes. Different symbols, such as circles and crosses, and different colored inks are used to identify the items recorded.

For computational purposes, hourly average values of each item are usually required. With some items, such as air temperature or wet-bulb temperature, the diurnal change is reasonably regular, and hourly averages can be determined quickly and accurately using a graphical technique. For the computation of solar and atmospheric radiation, three items must be recorded, namely: (1) a voltage proportional to the solar radiation received by the pyrheliometer, (2) a voltage proportional to the total radiation received by the radiometer, and (3) a voltage proportional to the temperature of the radiometer plate. On clear days and on some totally overcast days the diurnal variation in these three items is also regular, and hourly averages can be determined quickly. On partly cloudy days, however, transient cloud effects cause rapid, short period variations of considerable magnitude in each item. If hourly averages are to be determined graphically, it is usually necessary to draw lines connecting consecutive points. Even so, the constructed trace is so erratic that the accuracy of the graphical average is questionable. Another procedure that has been used is to enter the ordinates of consecutive points in a desk calculator and compute the arithmetic averag

The frequency of partly cloudy days is sufficient to make manual data processing tedious and time-consuming. Experience has shown that in one week a technician can process the amount of chart recorded in 2 weeks in the

ield. The fact that the work is tedious should not be disregarded lightly; asks of this nature should be kept at a minimum to obtain maximum production from competent technical personnel. As the operations are repetitive and the field data can be put easily in the form of a digital record, the use of modern high-speed computing equipment was clearly indicated. Because a computer was available in the Washington office of the Geological Survey, it was decided to proceed with development of equipment that would record the data on punched paper tape in the field. The possibility of punching the tapes n the office was considered, but discarded, for the time required to transfer he data from the recorder charts to tape would be as great as hitherto needed to process the charts manually. It was considered inadvisable to eliminate the chart record. Visual inspection of that record often revealed short periods of questionable data resulting from equipment malfunctions. Inspection of the punched tape would not suffice, and it was therefore deemed essential that the data be recorded both on the chart and paper tape.

The field punching equipment consisted of three components, namely: (1) an analog-to-digital converter or encoder to convert the angular rotation of the shaft controlling the position of the potentiometer print wheel to a digital equivalent, (2) a programmer to insert certain computer commands and related information, and (3) a motorized punch to record the data and program commands on paper tape.

The first component, the encoder, was commercially available. The device divides the 11-inch width of the recorder chart into 889 finite parts and converts the information into binary-decimal code.

The second component, the programmer, was custom built. It would have been possible to punch only the basic data, a three-digit number giving an output voltage. The tape would then have to be repunched completely in the office n order to put the tape in a form usable in the computer. For example, for he Datatron 205 computer the data would have to be organized into 10-digit "words." Each channel requires identification. Programming instructions must be provided to identify the end of each hour and the end of each calendar day. The computational program for the meteorological data previously named is rather complicated; it was originally estimated that approximately two-thirds of the total storage capacity of the Datatron 205 would have to be reserved for the program commands, leaving only one-third of the storage for data. This would have been insufficient for a full day's data, so that the computer was ordered to read in data for a 12-hour period, make the computations, and deliver the results before starting another 12-hour period. Later t was found that the program required much less storage space than had been anticipated, and the schedule has been modified accordingly.

The programmer is necessarily custom built if the punched tapes are to be used directly in the computer without resort to extensive tape preparation. However, it does not seem wise to construct a large number of programmers hat will produce tapes usable in only one computer. Computer design is not static; tremendous advances have been made in recent years and more are to be expected. The expense of repunching the tape in the office prior to computing must be weighed against the cost of modifying a number of field installations if the computer requirements should change.

For the third component, a commercially available punch is used. The only modification required to make the punch suitable for unattended operation for periods as long as a week was to provide a motor-driven takeup reel for the punched tape and to provide a receptable for the "confetti" resulting from holes being punched in the tape.

In an evapotranspiration research project in Nebraska in 1958 the field data were recorded on three multi-channel recording potentiometers and at the same time punched on paper tape. The data from an 8-channel recorder have been computed on a Datatron 205 at Denver Research Institute. It was found that it took approximately 1 hour to compute one week's data, as compared to 2-1/2 work days required for manual data processing.

Some possible modifications of the computing program are under consideration. The actual time required for the computer to make certain computations seems to be rather long. Preliminary studies indicate that it might be advisable to compute the radiation items from daily averages rather than from hourly averages. Over the diurnal range usually encountered, many of the relationships are nearly linear, and the error resulting from using daily averages appears to be negligible. A critical review of the complete program indicates that certain revisions will result in a substantial reduction in computing time. Modifications in the recording and computing scheme have already been made that will permit computing a week's data instead of 12 hours at a time.

CONCLUSIONS

The advantages of using digital computers for hydrologic studies are many In studies using streamflow records, the computations are not particularly complicated, but the mass of data is tremendous. The use of digital computing equipment will permit much better utilization for statistical studies of the great store of streamflow data accumulated over the years. In research studies not as many data are involved, but the computations may be somewhat more complicated. For all types of hydrologic data obtained by the Geological Survey, the trend is toward obtaining a record in the field on tape which can be used directly or automatically converted for use in a digital computer.

With high-speed computers the results can be available much sooner than if the computations are made manually. For various reasons provisional records of discharge at some gaging stations are now computed currently. With digital computing equipment it might be practical to increase the number of such records substantially, for with this equipment the cost of recomputation at the end of the water year (which is almost always necessary) might not be excessive.

Another advantage resulting from the use of digital computing equipment for hydrologic research is the ease with which the basic data can be placed in a form suitable for publication. The basic data for some research projects have not been published simply because of the labor involved in preparing the tables and setting them in type. Modern computing equipment can produce tabulations of data and results of computations that are suitable for offset reproduction. Column headings and rulings can be added easily using transparent overlays.

Perhaps the principal advantage of modern computing equipment accrues from more efficient utilization of technical personnel. Much of the data processing and many of the computations required for hydrologic studies are both monotonous and tedious. Reducing the amount of time that trained personnel must spend on purely routine tasks will pay dividends in more comprehensive and thorough analyses of the data.

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HYDRAULIC DOWNPULL FORCES ON HIGH HEAD GATES[a]

Donald Colgate,[1] M. ASCE

ABSTRACT

The paper discusses model studies in the Bureau of Reclamation's Hydrau-ic Laboratory in which both the direct weighing method and the pressure area computation method were used to determine hydraulic downpull on two high ead gates, and one model study in which the pressure area computation nethod alone was employed. Also described are the methods used and re-ults obtained in the field measurements of the prototype installations.

INTRODUCTION

The general design of a gate hoist provides for sufficient capacity to open r close the gate under full operating head. The required capacity of the gate oist is usually obtained by totaling the frictional resistance of the gate, the dead" weight of the gate parts being moved, and the hydraulic forces tending o open or close the gate. The weight of the gate and other moving parts, ither dry or submerged, is easily computed. The frictional resistance can e computed by using assumed coefficients of friction. However, the hydrau-ic force caused by the unbalanced water pressure above and below the gate eaf, particularly when the gate is in the throttling position, is far from imple to determine. A few tests to determine these hydraulic forces are resented here.

Water passing beneath a gate leaf creates a reduced pressure due to a hange in the direction of flow and a change in energy from pressure head to elocity head. This reduced pressure acts on the bottom of the gate. There s a pressure on the top of the gate, water pressure if the leaf is submerged, nd atmospheric pressure if it is above the free water surface. The downward

ote: Discussion open until April 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2245 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 11, November, 1959.

[a] Presented at the June 1958 ASCE Convention in Portland, Oregon.

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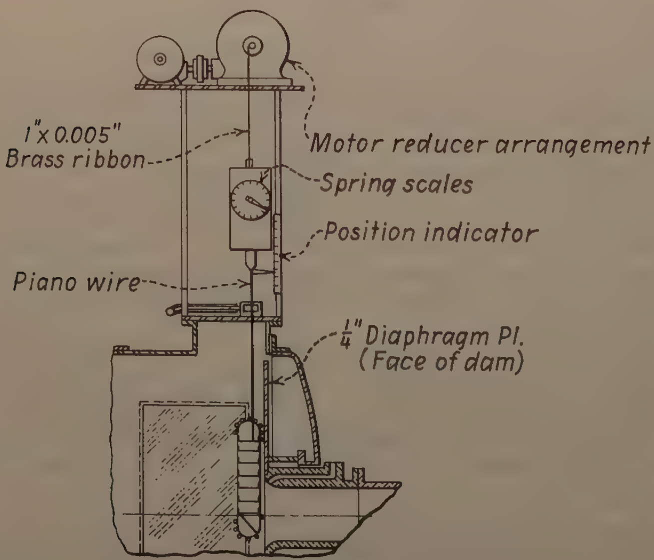
hydraulic force on the gate is equal to the difference in pressure above and below the gate in pounds per square inch multiplied by the affected cross-sectional area of the gate in square inches. By changing the shape of the bottom of the gate, or by altering the flow passage, a pressure change might be achieved which would reduce the hydraulic downpull. However, each new gate installation offers a new and specific problem in regard to hydraulic downpull, and each must be dealt with individually.

Shasta Dam Fixed-wheel Gate

Laboratory studies have been made to determine the hydraulic downpull on several different types of gate installations. One of the Bureau of Reclamation's earliest model studies concerned an emergency bulkhead-type gate for Shasta Dam, designed to close the inlet to a conduit discharging under 323 feet of head. An attempt was made to "weigh" the model downpull forces by suspending the gate from a spring scales and taking continuous weight readings while the gate was being raised or lowered (Fig. 1). However, the hydraulic downpull values were extremely difficult to determine because of excessive and variable frictional forces. The roller chains were removed from the initial model gate and replaced with small flanged wheels resemblin[g] miniature railroad wheels. Under heads up to 10 feet the apparatus seemed to operate satisfactorily, but for higher heads the wheels would stick momentarily and release suddenly, producing the same untenable conditions encountered with the roller chains. A new model gate was fabricated using wheels having a diameter equal to the thickness of the gate. These wheels



DETAIL OF MOTORIZED GATE LIFT

Figure 1.

urned on ball bearings to minimize the rolling and bearing friction. Nevertheless, the movements of the gate were still so unsteady the forces were extremely difficult to record. It was found that the roller tracks had to be made glassy smooth before acceptable readings could be made.

Several strategically placed piezometer taps were installed in the model gate and the downpull forces were computed by the pressure differential-area method. Results of the pressure measurements for various gate bottom designs are shown in Fig. 2.

From these tests, it appeared that the extreme precision required in fabricating this type of a model gate to be acceptable for weighing the downpull forces overshadowed the tedious recording and computations required in a pressure study.

The gate developed in the above studies was installed at Shasta Dam, and an opportunity arose to measure the downpull forces in the field installation. An SR-4-type strain gage was bonded to the gate hoisting stem and the change in resistance of the strain gage was measured by a portable strain indicator. The gate position was measured with an engineer's chain. As the test proceeded, the strain indicator, the engineer's chain, and a timing device were photographed by a motion picture camera operating 3 frames per second. The timing device was included to correlate the gate opening and strain readings with a metering device. A chart showing the hydraulic downpull determined by a model study using the pressure area method and by field measurements is shown in Fig. 3. The discrepancy between model and prototype at about 20 percent gate opening is perhaps due to improperly computing the

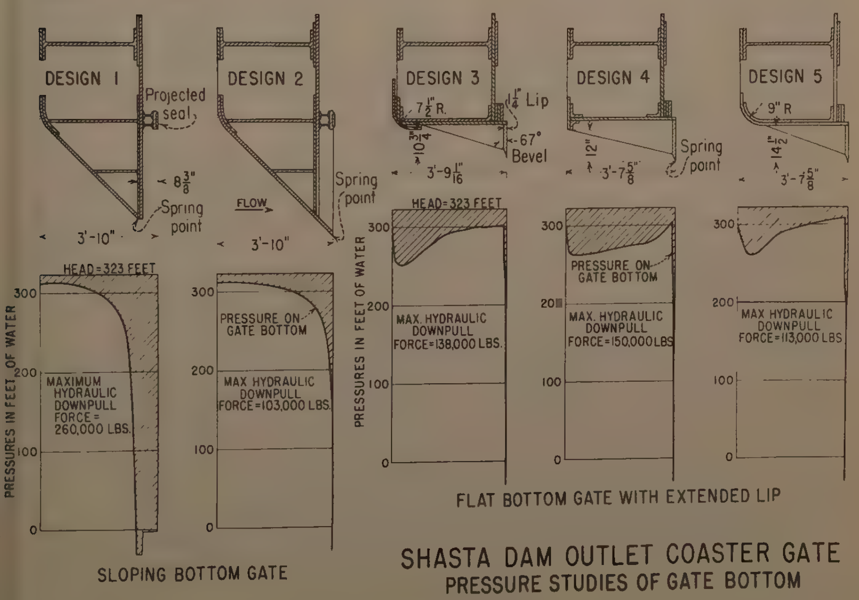


Figure 2.

downpull contributed by the upper seal as it enters the area of the recess in the face of the dam.

Hoover Dam Cylinder Gate

A laboratory study was made of the hydraulic downpull forces on the cylinder-type gate at Hoover Dam. Since the horizontal forces acted radially inward and were resisted internally by the gate, bearing and rolling friction were not a factor in the study; force measurements by weighing appeared to be the simplest method of solution.

The cylinder gates were suspended by molybdenum-bronze wires in a 1:24 scale model of one of the intake towers (Fig. 4). The change in downward forces on the gate could be computed from the measured change in strain in the wire.

At openings of 20 inches and larger, the reduced pressures on the bottom of the gate were due mainly to the change in direction of the flow. In this range, a small change in gate opening was accompanied by only a slight change in the hydraulic downpull, and the model apparatus was quite stable, producing reliable results. As the gate neared the closed position, the area of the flow passage between the surface on the bottom of the gate and the gate seat increased in the direction of flow producing a venturi effect which tended to further reduce the pressure on the gate bottom. At these smaller gate openings, a slight change in gate opening or a small lateral displacement of the gate between the guides would change the shape of the flow passage under the gate and produce a large change in pressure on the bottom of the gate. These pressure changes would be either uniform around the periphery of the gate or unbalanced, and the gate would tend to move in the direction of the lowest pressure. The gate movement would be resisted by the suspending mechanism or be arrested by the gate guides. As the gate moved, the distribution of forces would shift rapidly to a new location, and the entire model, although sturdily constructed, would shake so violently that the gages could not be read. No practical solution to the problem was found, although many

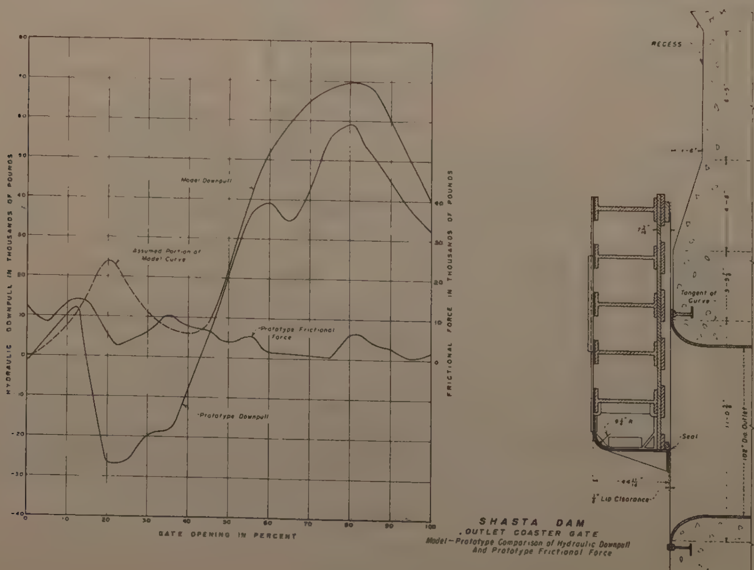


Figure 3.

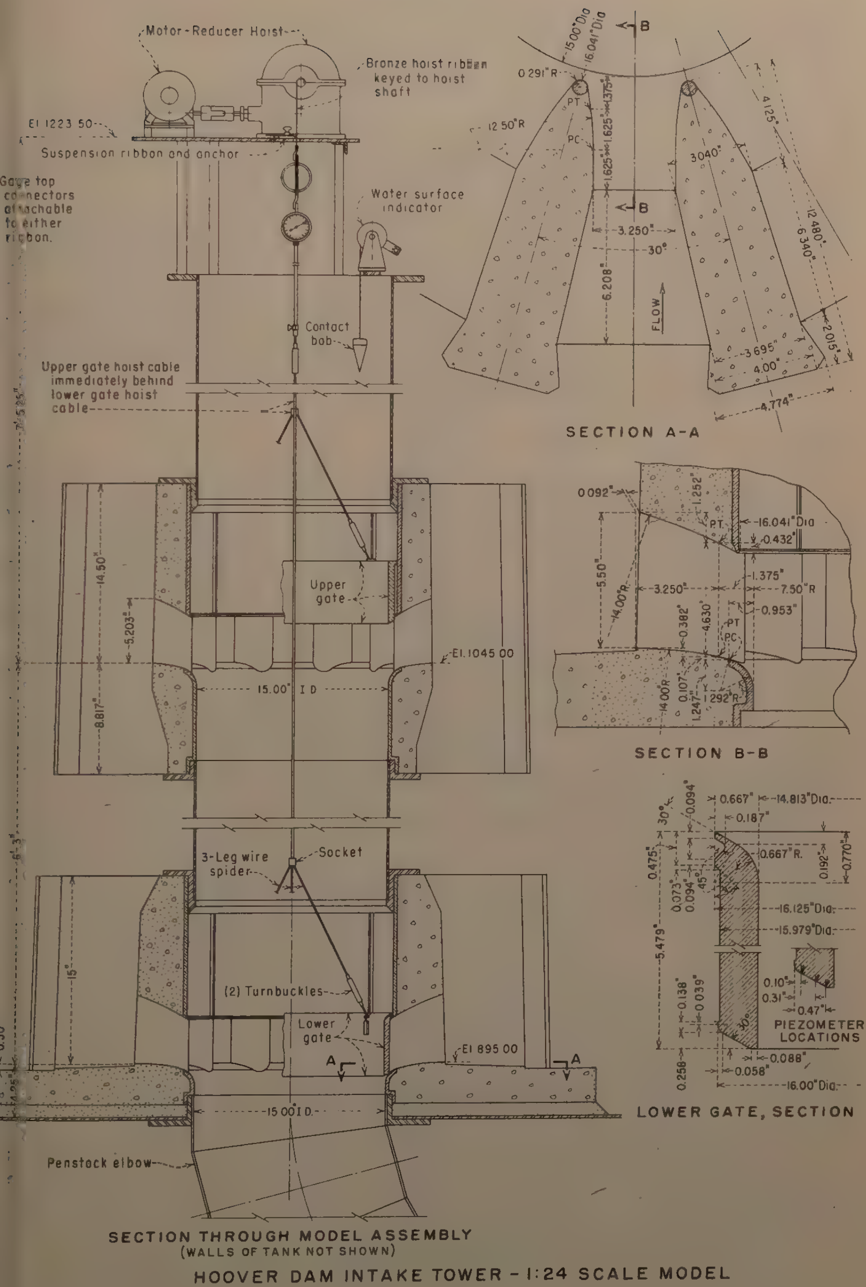

Motor-Reducer Hoist
Bronze hoist ribbon keyed to hoist shaft
El 1223 50
Suspension ribbon and anchor
Gage top connectors attachable to either ribbon.
Water surface indicator
Contact bob
Upper gate hoist cable immediately behind lower gate hoist cable
Upper gate
El. 1045 00
15.00" I D
3-Leg wire spider
Socket
(2) Turnbuckles
Lower gate
El 895 00
15 00" I D.
Penstock elbow
SECTION A-A
SECTION B-B
PIEZOMETER LOCATIONS
LOWER GATE, SECTION
SECTION THROUGH MODEL ASSEMBLY
(WALLS OF TANK NOT SHOWN)
HOOVER DAM INTAKE TOWER - 1:24 SCALE MODEL

Figure 4.

corrective measures were tried. For the final tests, the gate was removed, piezometers installed, and pressure area studies were made with the gate wedged tightly in predetermined positions.

In the field, downpull tests were made on the Hoover Dam cylinder gate fo openings up to 15 inches. Prototype downpull was measured by means of strain gages on each of the three lifting nuts (Fig. 5). Continuous oscillograph records of the change in resistance of each gage were made during the tests, and from these records the downpull was determined. The downpull curve determined by these measurements followed the trend of the model curve but produced maximum values about 23 percent higher than those determined by model study, and the peak downpull occurred at 4-inch gate open ing whereas the model indicated a peak at 6-inch gate opening (Fig. 6).

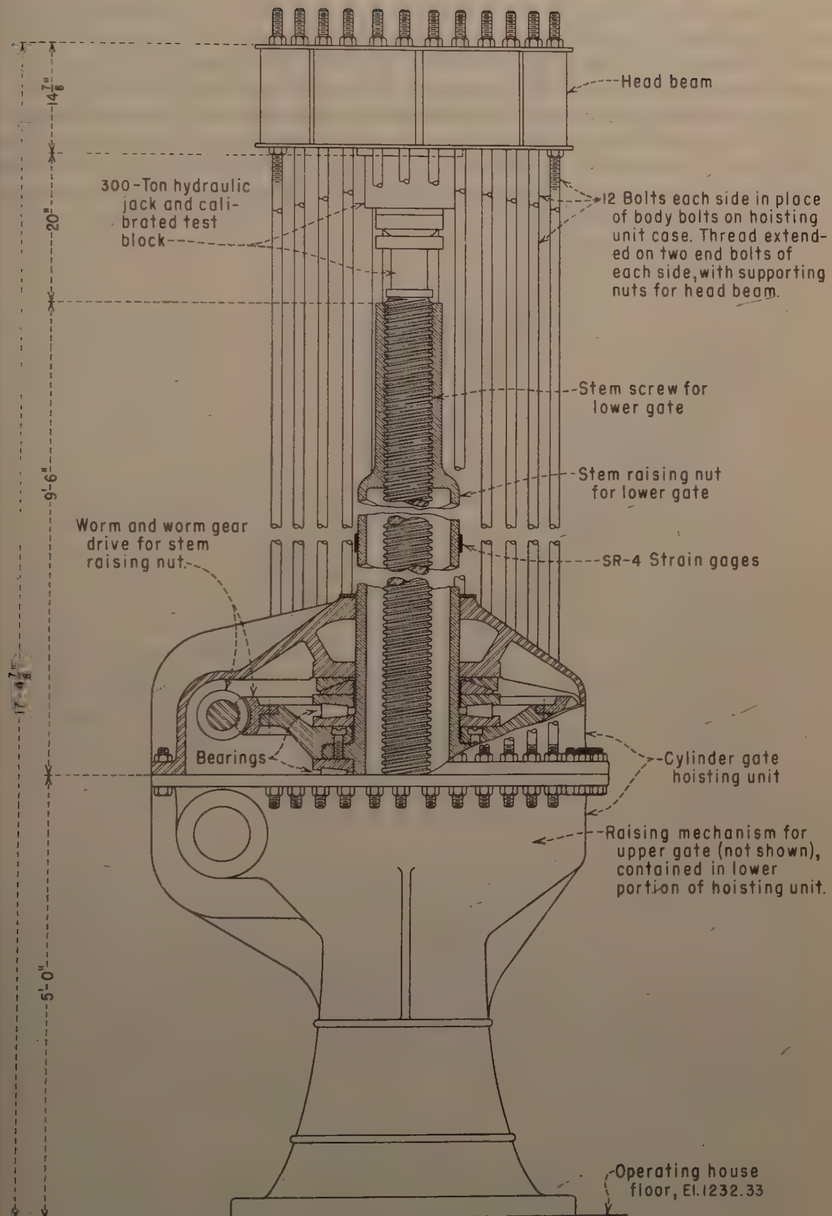
Obviously, the direct measurement of downpull forces on prototype gates is the most practical. However, a model to be used for direct force measurements must be made with such extremely confining tolerances that the economy of this type of study is questionable.

Palisades Fixed-wheel Gate

One of the preliminary plans for both the power tunnel and the outlet work tunnel at Palisades Dam considered the installation of a fixed-wheel-type gat covering a semibellmouth entrance, 20.0 feet wide and 39.3 feet high. A closed gate at this location subjected to a head of 156 feet, would be required to withstand a hydrostatic force of 7,980 kips. In another plan, a beam was placed across the opening above the tunnel entrance to form the top part of the semibellmouth (Fig. 7). This arrangement permitted the gate to be installed a short distance downstream from the tunnel entrance where the open ing was 19.7 feet wide by 28.0 feet high. The hydrostatic force on the closed gate at this location would be 5,540 kips, or a load reduction of about 30 percent. The latter design was preferable economically, and a model study was instigated to determine the hydraulic characteristics of the installation, particularly the hydraulic downpull on the gate.

In view of the previously mentioned studies, the weighing or force method of determining hydraulic downpull was not favorably considered for the Palisades model. Available data were analyzed to determine where critical pressure areas might occur on the gate so that an accurate pressure study might be made and the hydraulic downpull computed. Based on information thus obtained, 36 piezometer taps were placed in the 1:39 scale model gate (Fig. 8). The approach to the gate, the beam forming the top part of the semibellmouth entrance, and the transition downstream were formed of light weight sheet metal (Fig. 9); 24 piezometer taps were placed in critical point in the beam, entrance, gate slots, and transition. The entire apparatus was installed in a 36-inch-diameter pressure tank, and the 60 piezometer leads were carried through the wall of the tank and attached to a gage board.

Preliminary testing concerned the general operation of the system to mak certain the overall design would be hydraulically acceptable. When the gate was opened 80 percent or more and the discharge exceeded 18,000 cfs, the pressure on the downstream face of the gate exceeded that on the upstream face by a sufficient amount to force the gate away from the wheel tracks. This untenable situation was remedied by making a fairly large recess in the downstream face of the concrete beam (Fig. 7), thereby keeping a balanced pressure on the upstream and downstream faces of the gate. All other configurations appeared to be acceptable.



STEM NUT CALIBRATING APPARATUS FOR CYLINDER GATE HOIST - HOOVER DAM INTAKE TOWER

Figure 5.

Maximum discharge through the penstock was computed to be 45,000 cfs. Preliminary downpull tests disclosed that the forces prevailing with this discharge, and with the gate fully opened, were about as expected, producing hydraulic downpull forces of a little more than 700,000 pounds. However, a much larger downpull was encountered at about the midpoint of gate travel.

A study of the pressure distribution on the gate bottom disclosed that considerable variation in pressures occurred between adjacent piezometer taps. In computing the downpull, therefore, the small areas adjacent to each of the 15 taps were considered separately. The sum of these individual downward forces, together with the downpull attributed to the upper and lower seals, constituted the total hydraulic downpull for any given condition of gate opening and discharge.

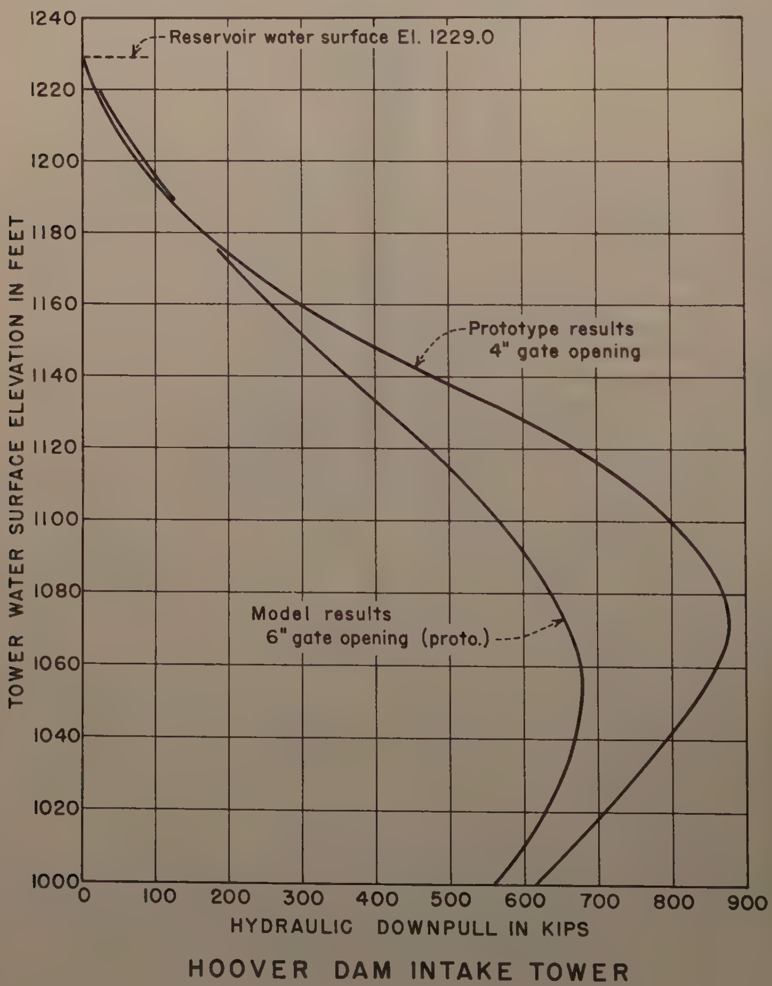


Figure 6.

If the emergency discharge of 45,000 cfs existed with the gate 100 percent open, and the gate then closed, the hydraulic downpull would vary as shown in Fig. 10. The maximum hydraulic downpull of 895,000 pounds at about 55 percent gate opening was peculiar to this particular gate bottom configuration; however, the 707,000-pound hydraulic downpull at 100 percent gate opening would exist for any gate with the same cross-sectional area regardless of gate bottom shape.

An extension of the downstream lip of the gate would create a higher pressure of the bottom web thereby reducing the hydraulic downpull; however,

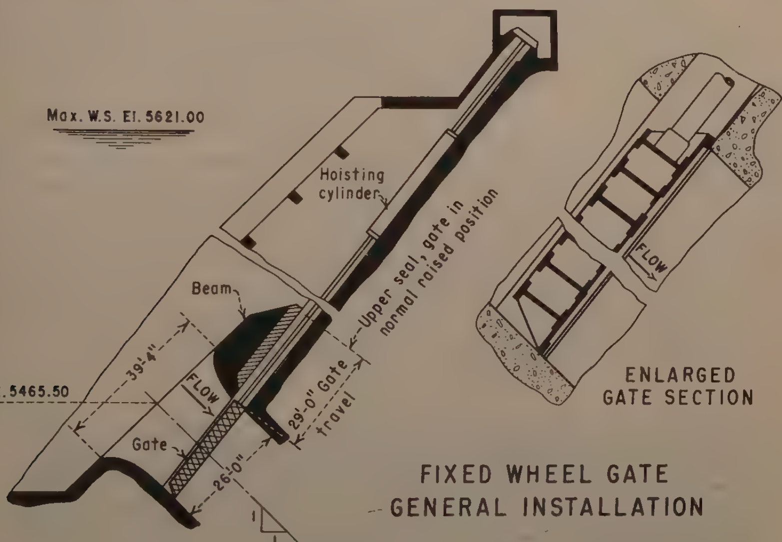


Figure 7.

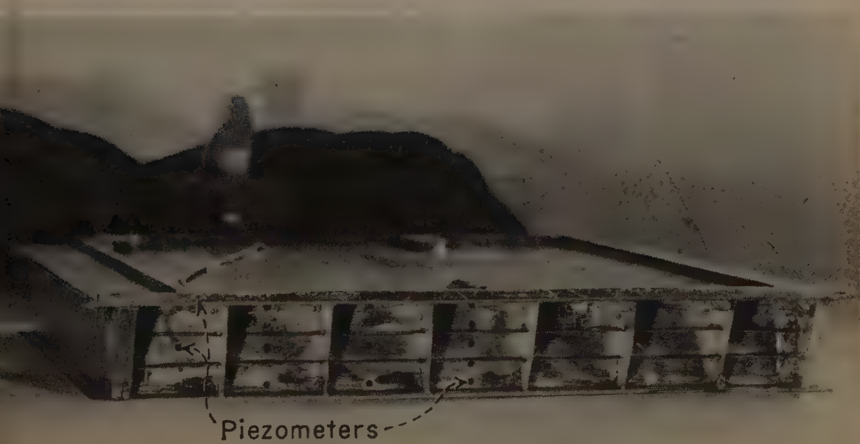


Figure 8.

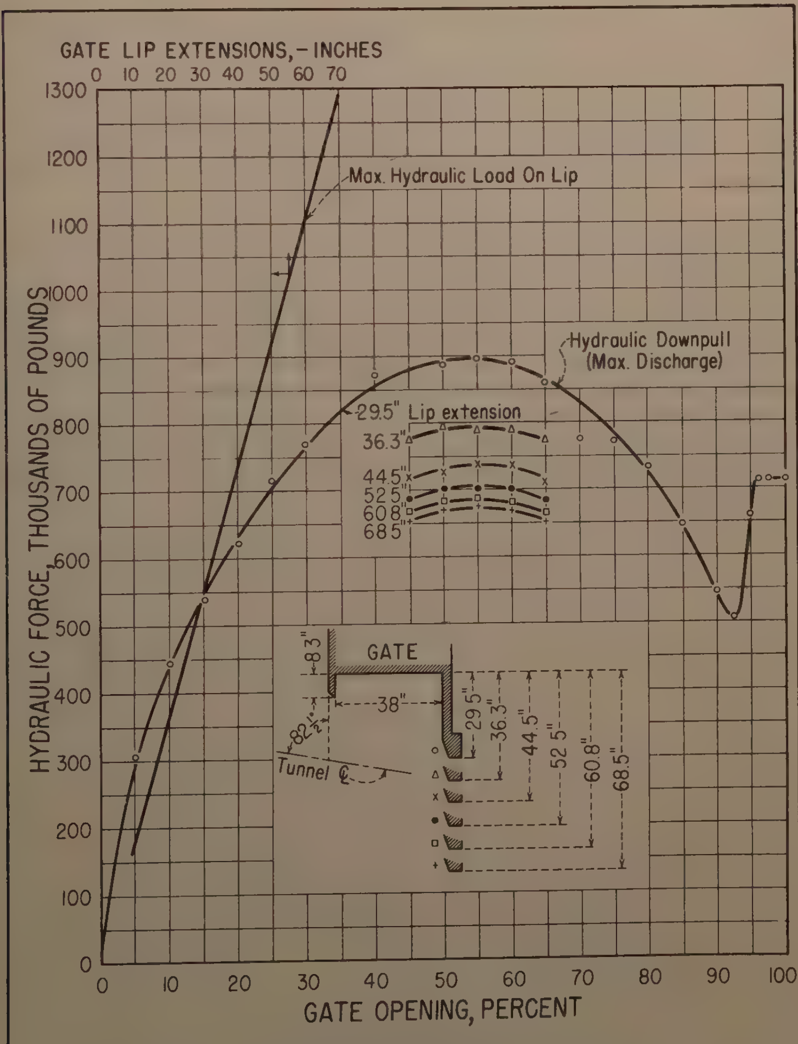
structural considerations limited the permissible amount of lip extension. The maximum hydraulic downpull for a 29.5-inch lip extension was 895,000 pounds, while that for a 68-1/2-inch extension was 670,000 pounds. The added cost of strengthening the 68-1/2-inch lip to withstand a thrust of 1,270,000 pounds (Fig. 10) was greater than the savings which would be realized using a smaller capacity hoisting cylinder. The designers, therefore, placed a limit of 550,000 pounds thrust on the extension, or a lip extension of about 30 inches.

Some reduction in downpull could be achieved by reducing the length of, and streamlining the upstream lip of the gate. The amount of streamlining or shortening was again a structural problem, so computations were made and the upstream lip changed accordingly. The configuration shown in Fig. 11, Design No. 2, was considered structurally sound and is the shape tested in the following study.

The fixed-wheel gate would be required to stop the flow caused by some emergency such as a ruptured penstock, control valve or valves rendered inoperative in the opened position, etc; 45,000 cfs is the maximum possible flow, and an air vent downstream from the gate limits the head here to 15 feet below atmospheric. The maximum reservoir elevation is 178.9 feet above the lower gate seat. Realizing these limitations, a calibration chart was made for the gate, plotting discharge against head drop from the reservoir to the air vent for various gate openings. Then, the pressures on the gate were measured and the hydraulic downpull computed for several

Figure 9.

discharges, and for each 5 percent increment of gate opening. Next, by cross plotting, a chart was drawn showing the hydraulic downpull which would be encountered as the gate was lowered from fully opened to closed, for any initial emergency discharge. As the gate was lowered into the stream, the discharge and the head at the air vent would decrease until the head at the air



EFFECT OF LIP EXTENSION ON DOWNPULL

Figure 10.

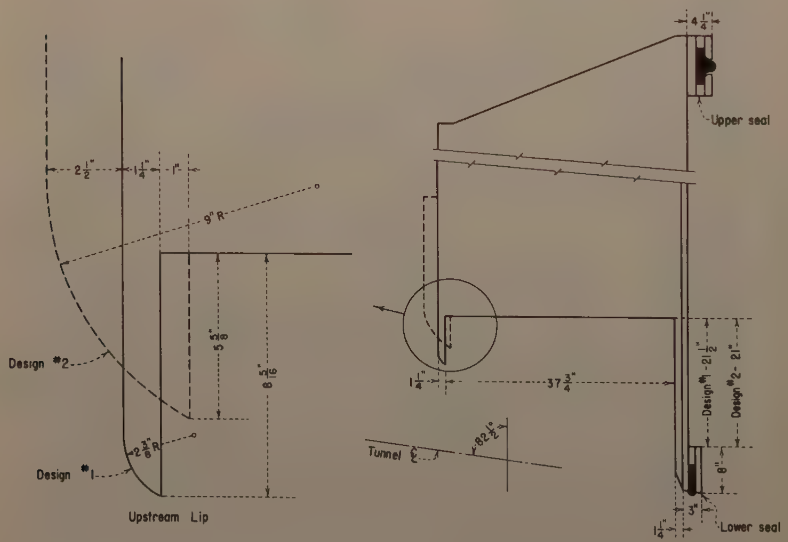
vent became negative 15 feet. At this point, maximum downpull would prevail for each initial discharge.

In the downpull chart, Fig. 12, the gate opening is based on a total travel of 29 feet; however, the gate does not affect the flow during the last foot or so of travel (above about 96 percent gate opening.) Therefore, the downpull is constant in this range.

The hydraulic downpull for maximum emergency discharge and the gate 100 percent open is 707,000 pounds, the same as for any other gate bottom design. The maximum hydraulic downpull is 813,000 pounds and occurs with the gate 57 percent open. The reduction in the upstream lip extension of 2-11/16 inches, comparable to extending the downstream lip this amount, would have reduced the maximum downpull by 40,000 pounds. The 9-inch radius on the upstream lip evidently reduced the maximum downpull by 42,000 pounds.

As a result of the hydraulic downpull tests, the hoisting cylinder and connectors were made sufficiently rugged to lower (or hoist) a total of 1,300,000 pounds.

A comprehensive field test concerning the hydraulic downpull at Palisades Dam has not been made. However, during a field operation for another purpose, the hoisting cylinder oil pressure was measured while the gate was moved from 100 percent open to 89.4 percent, and back to full open. The flow through the penstock at this time was 6,000 cfs. The results computed from this very limited study are shown in Fig. 12. The trend follows the model results very closely; however, the actual field values are about double those determined from model study.



GATE SEALS AND GATE BOTTOM SHAPES

Figure 11.

CONCLUSIONS

Not all of the parameters controlling the hydraulic downpull forces on gates have been defined to a degree that will permit an accurate mathematical analysis of each individual problem. Scale model investigations are of primary importance to the engineer in the solution of these problems. The two most common procedures employed in a model study are the direct force or

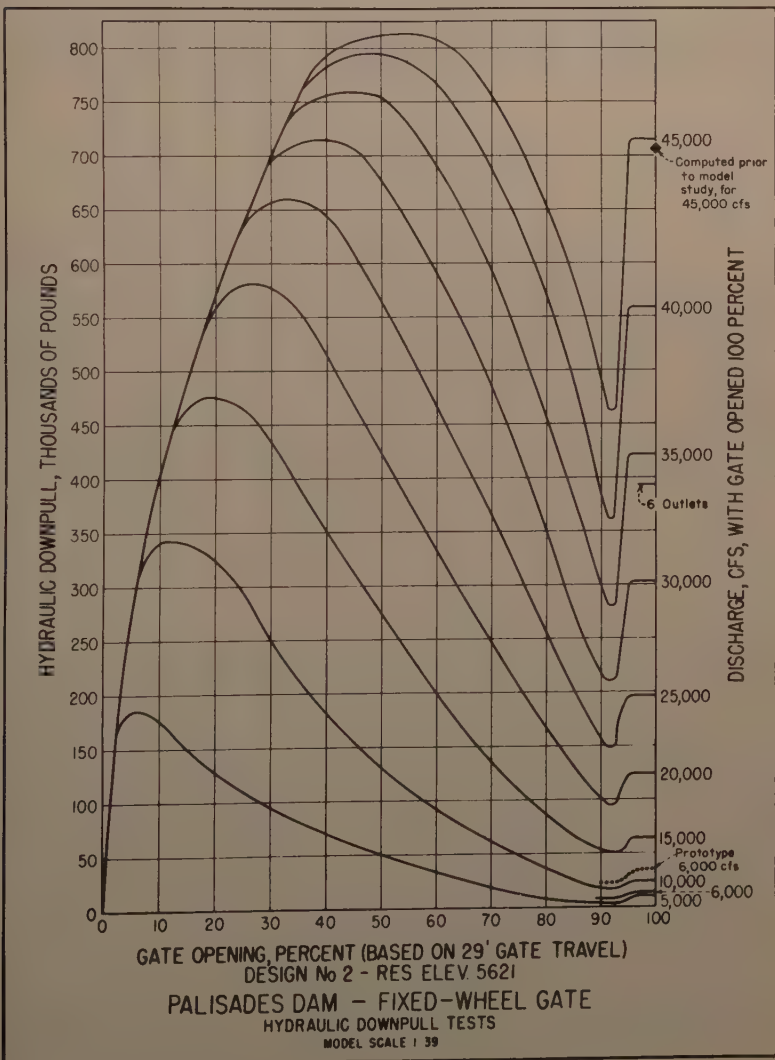


Figure 12.

"weighing" method, and the pressure area computation method. For most applied studies, the pressure area method is preferred because a simple and relatively inexpensive model will be adequate.

The moving parts of a model gate to be used for direct force measurement must be fabricated to extremely close tolerances. This type of model can be quite useful for basic research.

Field investigations should be made whenever possible, and the results compared to those determined by model study. A sufficient number of such correlations on a gate type will enable the engineer to understand more clearly the function of each variable contributing to hydraulic downpull. Then reliable charts and tables may be prepared from which the total of the hydraulic forces acting on this type gate may be computed.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

HYDRAULIC CHARACTERISTICS OF HOLLOW-JET VALVES

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SYNOPSIS

For free discharge conditions the hollow-jet valve has proved to be a satisfactory control device for flows through large conduits operating under high heads. Results of field tests with a 96-inch hollow-jet valve have revealed close agreement with hydraulic characteristics predicted from model studies. Piezometric measurements, thrust determinations on the valve needle, and rates of discharge were included in both field and laboratory tests. The prototype valve can be used as a metering device by the employment of model calibration results provided accurate position indicators are used. The only cavitation erosion evident in the prototype valve was caused by local irregularities in the body casting, which have been alleviated in subsequent valves by careful foundry practice and inspection.

INTRODUCTION

The search for a satisfactory valve to operate at any opening, to control flow through large conduits discharging under high heads, has been in progress for nearly half a century. The need for such a control device was responsible for development of the Ensign valve (Arrowrock Dam), the needle valve (Alcova Dam), and the tube valve (lower outlets through Shasta Dam). Other valves have also been developed for the same purpose, but the ones named constitute those primarily used by the Bureau of Reclamation. All of these valves had certain performance and economic limitations.

The increasing demand for closer control of releases through modern multiple-purpose structures prompted a continuation of studies to obtain a more suitable valve and led to development of the hollow-jet valve. This type, however, is limited to use as a free discharge valve preventing its

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application in closed conduits. The hollow-jet valve was developed by the Bureau of Reclamation and is patented (Patent No. 2,297,082) with rights reserved for use by the Federal Government without payment of royalties.

Models

The design was accomplished with the aid of three models; a 45 degree segment of a 12-inch-diameter air model, a 6-inch-diameter hydraulic model tested in the Hydraulic Laboratory of the Bureau of Reclamation at Denver, and a 24-inch-diameter model tested at Hoover Dam under a high head.

The prime purpose of the 24-inch model was to ascertain the hydraulic characteristics of a hollow-jet valve constructed under prototype conditions. That is, the outer shell, the supporting vanes, and the cylinder containing the needle were cast in one piece. Machine-finished surfaces were limited to the needle and that part of the outer shell flow surface upstream from the vanes. Because the rough finish of the casting could conceivably affect boundary flow sufficiently to cause local areas of low pressures, all surfaces in the 6-inch model were machine finished.

The secondary purpose for studies with the 24-inch model was exploration of some critical areas that were too small in the 6-inch model for exploration by piezometer orifices. A few revisions were found necessary as a result of tests with the 24-inch valve. This valve was later installed permanently in a Reclamation project.

Prototype

Although hollow-jet valves have been installed at a number of structures, initial installation of large units was on the four river outlets through Friant Dam, Fig. 1. Details of this installation are shown on Fig. 2. One of the four 96-inch valves was equipped with piezometer orifices to permit performance of special tests to ascertain if this type valve possessed predicted hydraulic characteristics. Fig. 3 shows locations of piezometer orifices in the valve. Where distances between orifices were small in the direction of flow, the orifices were offset laterally. A photograph taken looking upstream at this valve is shown on Fig. 4. Performance of the large valve could conceivably differ from that predicted by the 6-inch and 24-inch hydraulic models due to roughness of the large casting and because of larger tolerances necessarily allowed for machined surfaces in the prototype valve.

A field testing program was inaugurated about a year after the valves were placed in operation. The program consisted of piezometric measurements at valve openings of 10, 20, 40, 60, 80, and 100 percent of needle travel with operating heads of 102, 180, and 223 feet. The maximum design head at this installation is 246 feet. For each test, discharge through the valve was obtained from operating records based on current meter measurements in the river channel a short distance downstream from Friant Dam. In this instance all flow in the river passed through the valve under test. Hence, measurements of river flow were not subject to possible errors due to subtracting flow from another source such as a powerhouse.

Other field observations included general characteristics of the hollow-jet valves; such as, noise intensity and vibration, inspection of the interior of one unit, and stability of the jet.

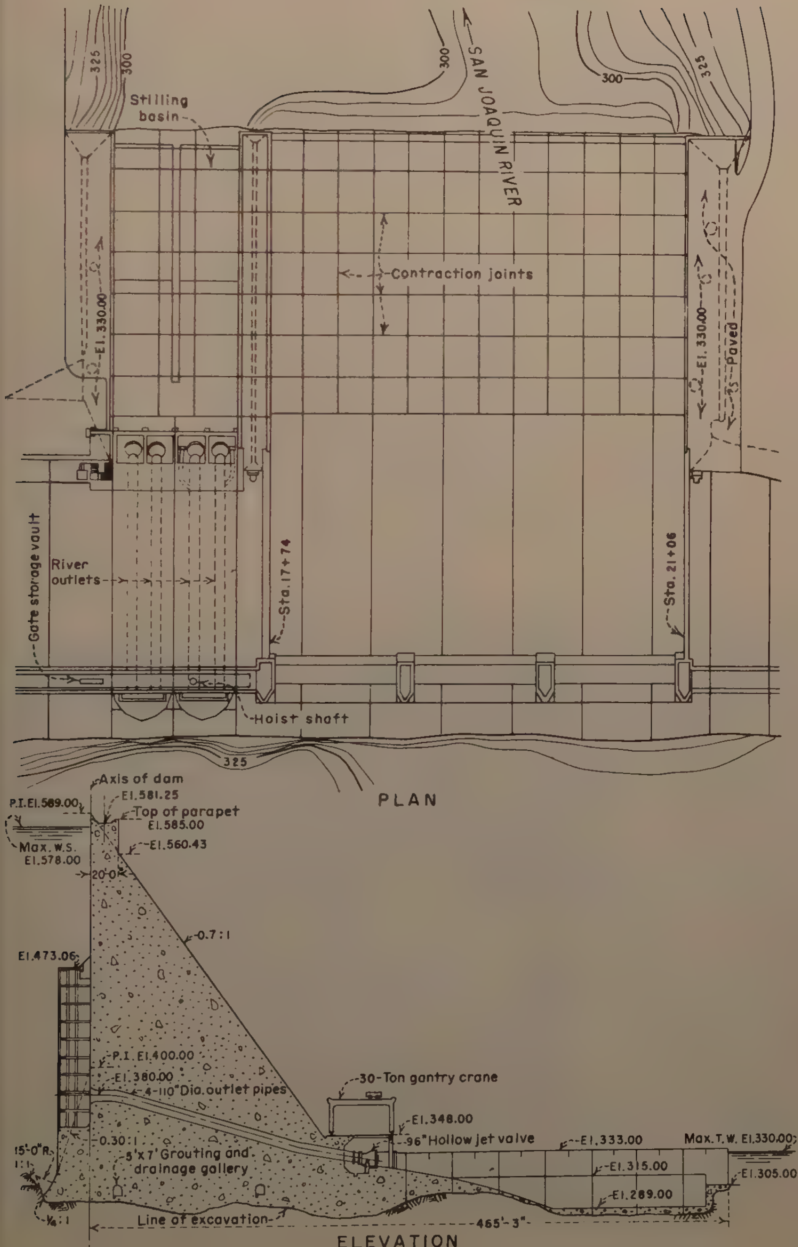


FIGURE 1 – PLAN AND ELEVATION OF RIVER OUTLETS
FRIANT DAM

Results of Pressure Measurements

Pressure measurements were obtained by connecting piezometer orifices to manifolds joined to mercury gages. A valve on each connecting line permitted determination of pressure for any particular piezometer. All pressures were referred to the elevation of the center line of the upstream end o

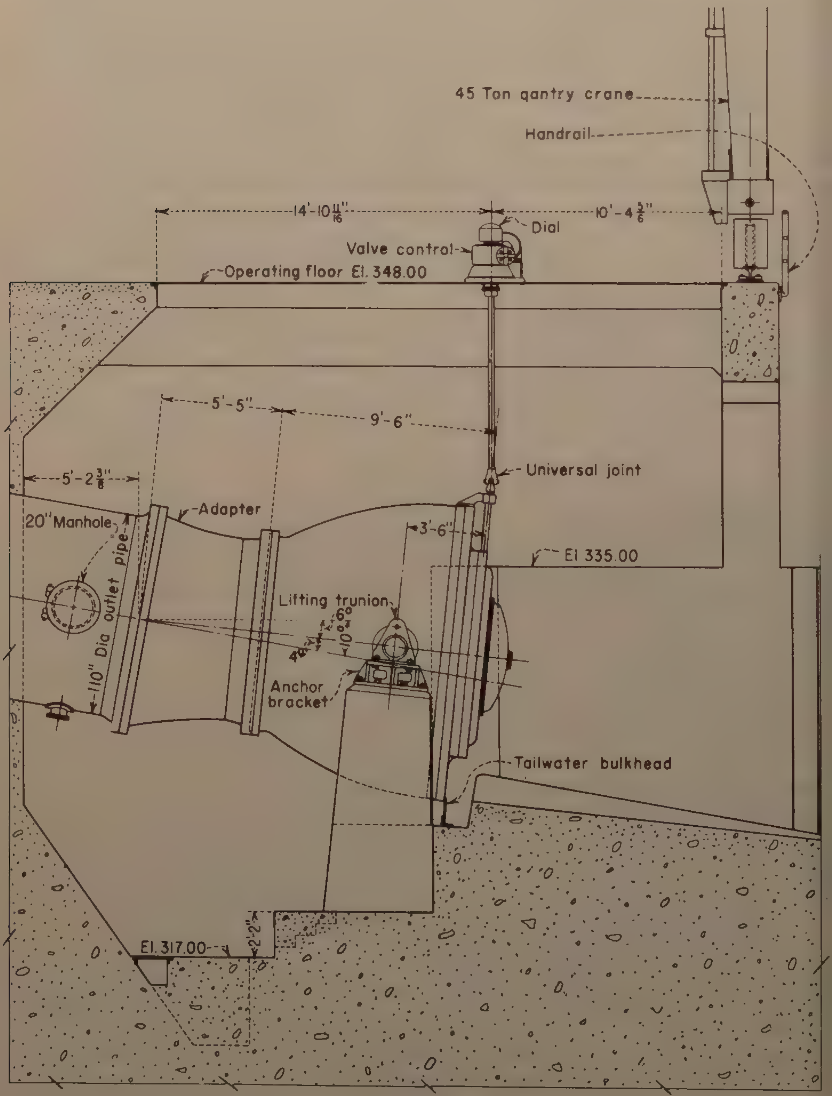


FIGURE 2 - 96-INCH HOLLOW JET VALVE INSTALLATION

FRIANT DAM RIVER OUTLETS

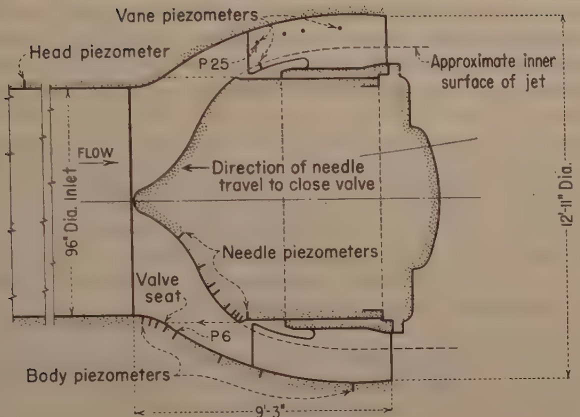


Figure 3 - Piezometer locations in 96-inch hollow-jet valve--Friant Dam River Outlet

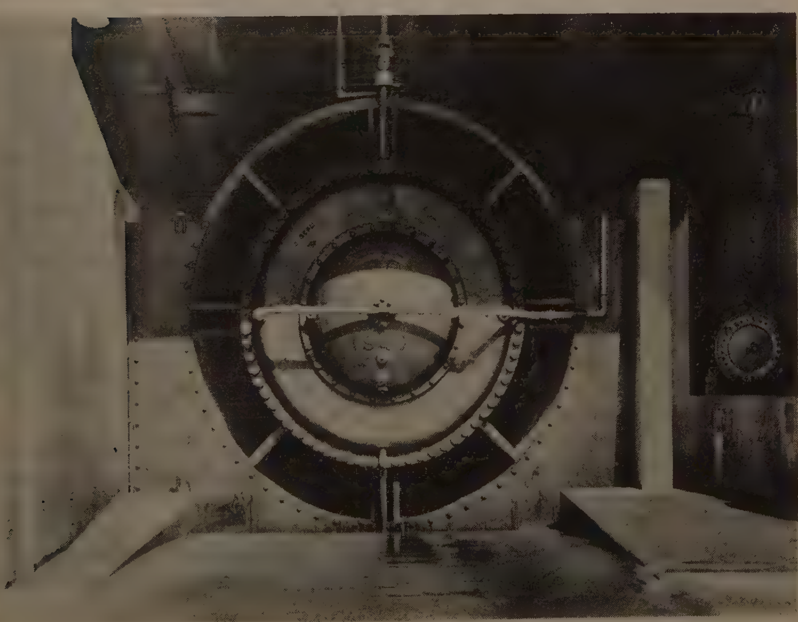


Figure 4 -Two-rack conduit (horizontal) encasing piezometer leads from interior of 96-inch hollow-jet valve--Friant Dam River Outlet

the valve. The valve operating head was measured with a piezometer orifice one inlet diameter upstream from the valve and referred to the same elevation as the other pressures. Hence, the prototype valve was considered to have a horizontal center line that corresponded to the hydraulic models, while actually the center line of the prototype valve sloped downward as shown on Fig. 2.

Pressures were plotted against percent valve opening by utilizing a pressure factor, F, defined as the ratio of the measured piezometric head, in feet of water, to the total head (static head plus velocity head), one inlet diameter upstream from the valve, Fig. 5. This procedure reduces F to a dimensionless ratio and makes it possible to obtain the pressure for any head at any piezometer in the valve by selecting from plotted pressure curves the corre value of F and multiplying it by the total design head on the valve one diameter upstream from the inlet.

As an example, to find the pressure at one of the piezometers, for which plot is shown on Fig. 5, when the total design head is 200 feet of water and the valve is 50 percent open, follow the 50 percent line on the plot until it intersects the curve for the particular piezometer and read the value of the pressure factor at the left. Then multiply the pressure factor by 200 to obtain the pressure at the piezometer for the case being considered. If the piezometric pressure is below atmospheric, then the F value is negative and the calculated pressure is also negative.

Plots of pressure factors versus valve openings, similar to Fig. 5, were made for all piezometer orifices installed in the 96-inch valve and in the models. The locations of Piezometers P6 and P25, for which pressure plots are shown on Fig. 5, are shown on Fig. 3.

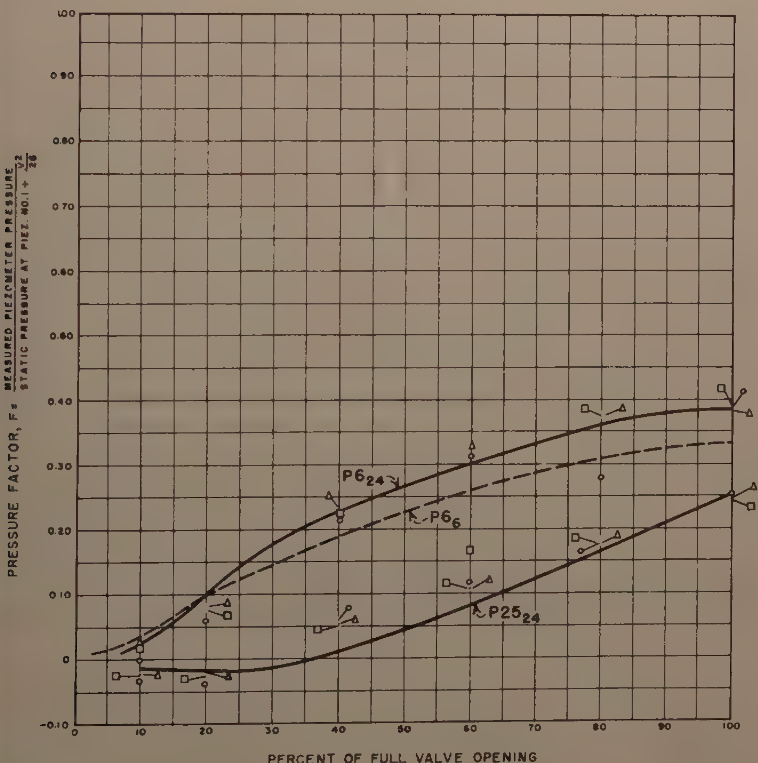
The pressure at the needle piezometer in the air space just upstream fro the vanes was found to be a negative 1.22 feet of water in the 24-inch model when 100 percent open under a total head of 196.6 feet. In the prototype, the corresponding pressure when referred to the same total head was a negative 4.9 feet of water based on pressure factors obtained from tests at the two highest reservoir heads. Based on the pressure factor from the test at the lowest reservoir elevation, the pressure was a negative 9.8 feet of water whe referred to the same total head. All of these values were obtained with a valve opening of 100 percent. Variation between model and prototype, as wel as variation in the prototype itself, is attributable to the fact that this region is filled with an air-water mixture due to insufflation of the jet in the prototype valve. This mixture could conceivably choke the air supply sufficiently to cause an increase in subatmospheric pressure, while in the model valve, little, if any, insufflation occurred.

The only subatmospheric pressures predicted from model studies were o the large vanes, but these were not considered sufficient to produce cavitatio erosion. This conclusion was verified by the prototype tests which revealed an average pressure of minus 13 feet of water in this region, but no pressure conducive to cavitation occurred. However, negative pressures were found i other locations in the prototype valve contrary to model measurements. The magnitudes of such pressures were small and insignificant.

In general, pressures measured in the 96-inch prototype valve differed from those measured in the 24-inch model by an amount approximately equal to the difference between the values determined in the 6-inch and 24-inch models. The average deviation between model and prototype pressures were found to be less than 10 feet of water at maximum prototype test head. These

viations can be attributed to a slightly different location of piezometer ori-
es, interference by a bolt head near a piezometer orifice, a slight dif-
rence in contour of the flow passages, or insufflation of portions of the
ototype jet.

As previously stated, no measured pressures were sufficiently low to
use cavitation erosion. However, inspection of the tested prototype valve
vealed that cavitation erosion had occurred on several localized areas of
e valve body upstream from the vanes. Although erosion of the affected
eas was not severe, the metal had been pitted. Since there was no estab-
hed pattern or zone of cavitation damage, this condition was attributed to



SYMBOLS
O RESERVOIR ELEVATION 433.98
□ RESERVOIR ELEVATION 512.32
Δ RESERVOIR ELEVATION 554.40

NOTE
Prototype data shown by symbols
24-Inch model data shown by solid lines
6-Inch model data shown by broken lines.

PIEZOMETERS 6 AND 25

FIGURE 5 – HYDRAULIC PERFORMANCE TESTS
FRIANT DAM RIVER OUTLET VALVE

the rough surface of the casting. The fact that cavitation damage did occur exemplifies the need for specifying cast surfaces with a roughness factor held within specified limits.

At the time of inspection, the valve had operated for a total of 5,896 hour at openings varying from 2 to 66 percent under heads from 78 to 207 feet. Most of the operating time had been at openings less than 30 percent and heads less than 200 feet.

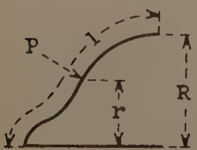
Results of Thrust Measurements

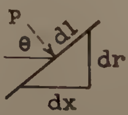
During design studies, considerable emphasis was placed on location and size of openings or ports through the needle portion of the valve to admit pressure into the interior, thereby balancing pressures so as to minimize power required to open and close the valve. Fig. 6 shows thrust on the nee in the upstream and downstream directions predicted from the 24-inch mod together with comparable information obtained from the 96-inch prototype.

For comparison purposes, the units on Fig. 6 have been reduced to those applicable to a 1-foot-diameter valve under a 1-foot head. This permits computation of thrust forces for other size hollow-jet valves operating unde various heads. These computations can be made by multiplying results sho on Fig. 6 by both the desired head and the square of the diameter of the val under consideration, these values being in feet and square feet, respectivel The results of similar data obtained on the 6-inch model are not shown sinc locations of the balancing ports established by tests on this small model we changed after analysis of results from the 24-inch valve.

The values shown on the plot reveal very close agreement between thrus forces predicted from the 24-inch model and those determined by field mea urements on the 96-inch valve. The greatest difference between model and prototype results occurs in downstream thrust at a valve opening of 10 per-cent where the prototype value is approximately 94 percent of that determin in the model study. The unbalanced force or the difference between the up-stream and the downstream thrusts is the most important, and the maximu unbalance occurs at a valve opening of 100 percent. This unbalanced force 1.29 times the value predicted from the model.

The thrust on the needle in the downstream direction was computed from prototype data as follows:





Let P = measured piezometric pressure
l = length along surface of needle
r = radius to piezometer
$2\pi rPdl$ = total thrust on incre ment dl
$2\pi rPdl \cos\theta$ = thrust on in-crement dl in x - directi

Total thrust in x-direction $= 2\pi \int_0^l rP\cos\theta\, dl$ and since

$dl = \dfrac{dr}{\cos\theta}$, then $2\pi \int_0^R Prdr$ = total thrust.

The integration was done graphically since relationship of P to r was t determinable analytically. Values of Pr as determined with piezometers the needle were plotted against r values for a valve opening of 10 percent, d the area under the curve was obtained with a planimeter to obtain

R
Prdr and that value of area was multiplied by 2π to obtain total thrust on

e needle in the downstream direction. The same procedure was utilized for lve openings of 20, 40, 60, 80, and 100 percent.

Thrust in the upstream direction is the pressure inside the needle, which s measured, multiplied by the area over which the pressure acts.

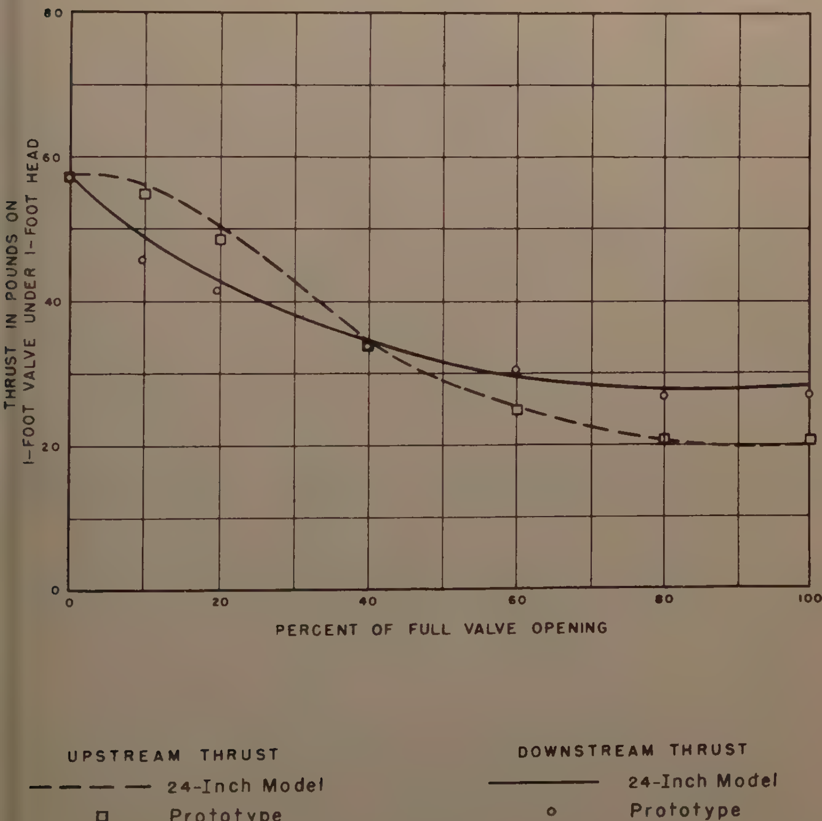


FIGURE 6 - THRUST ON HOLLOW JET VALVE NEEDLE
FRIANT DAM RIVER OUTLET

Rate of Discharge

Comparison of rates of discharge of a prototype and model valve are par ticularly important since they serve to evaluate discharge curves prepared from laboratory calibrations. The use of model instead of field calibrations can result in saving large amounts of money and time. The time and expens involved in performing a field calibration are demonstrated by the fact that approximately 600 current meter traverses were made over a period of 5 years to determine discharges through the river outlet valves at Friant Dar At this same structure, similar current meter measurements were perform to determine discharges through the hollow-jet valves at the headworks of Friant-Kern Canal, and also through needle valves at the headworks of Mad Canal. Results show that the current meter measurements could have been dispensed with since discharge curves established by model calibrations we as accurate as curves determined by field measurements.

Once a laboratory calibration has been made of a valve, this same calibr tion may be utilized for all installations of the same valve except for certai situations where complicated approach conditions disrupt flow characteristi

Figs. 7 and 8 present data to support accuracy of the laboratory calibra- tion curves. For valve openings greater than 15 percent, the variation be- tween model and prototype discharges may be considered as 3 percent. Fo smaller valve openings the difference is greater and may be partly account for by the fact that lower discharges in the prototype were not susceptible to accurate measurements with current meters.

When using valves as metering devices extreme care is necessary to in- sure that position indicators accurately reveal true valve openings. The particular valves described in this paper are equipped with verniers on the

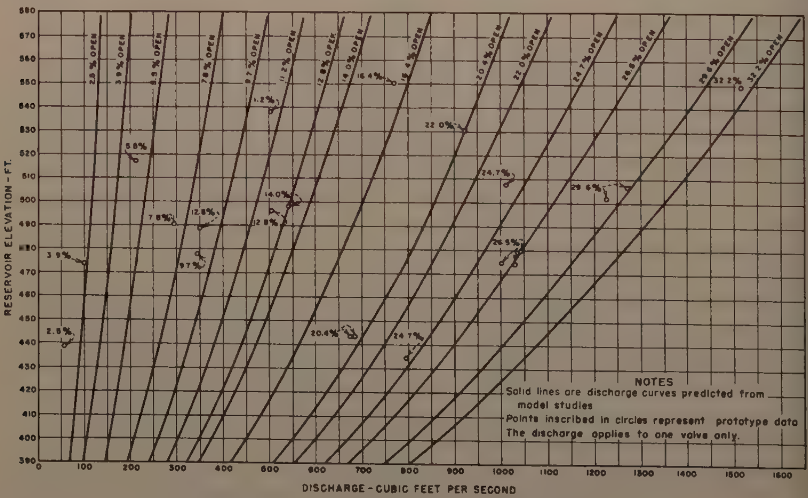


FIGURE 7 - HYDRAULIC PREFORMANCE TESTS
COMPARISON OF DISCHARGE BETWEEN MODEL AND PROTOTYPE
FRIANT DAM RIVER OUTLET

osition indicators to permit accurate setting of valve openings. An equally nportant item is that the true head on the valve must be known. Difficulties ave been encountered in some instances due to an improperly operating head age.

General Behavior of the Valve

General behavior of the hollow-jet valves has been entirely satisfactory. 'his statement applies to both mechanical operation and hydraulic charac-eristics. Operation has been quiet, free from vibration, and the jets remain table and well defined.

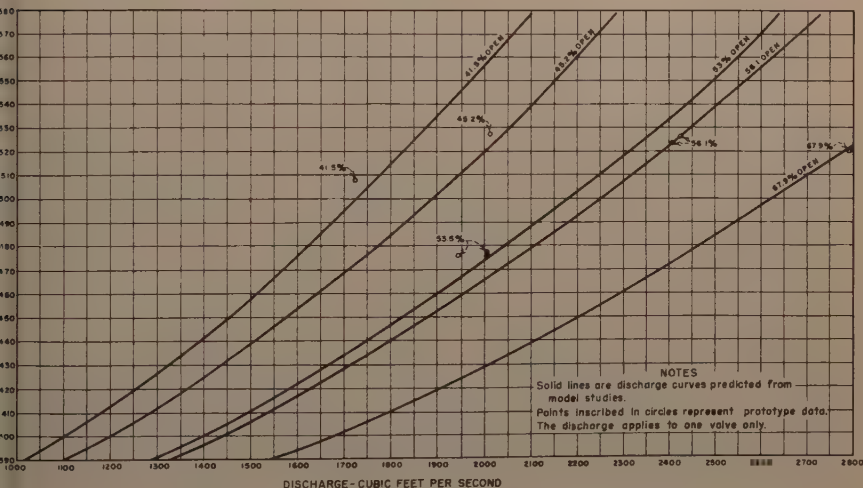


FIGURE 8 – HYDRAULIC PREFORMANCE TESTS
COMPARISON OF DISCHARGE BETWEEN MODEL AND PROTOTYPE
FRIANT DAM RIVER OUTLET

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

DISCHARGE FORMULA FOR STRAIGHT ALLUVIAL CHANNELS[a]

Hsin-Kuan Liu,[1] M. ASCE and Shoi-Yean Hwang,[2] A. M. ASCE

ABSTRACT

The proposed discharge formula contains a discharge coefficient and exponents for hydraulic radius and slope. Both the coefficient and the velocities are given as functions of bed forms and bed material. Velocities computed by the formula check closely with those obtained in the laboratory.

I. INTRODUCTION

The original title of this research was "Analytical Study of Alluvial Channel Roughness", which was a research project granted to the first author by the National Science Foundation. The purpose of this research was to find a suitable formula to determine more accurately the mean velocity of flow, and thereby the discharge of water, in alluvial channels. At the beginning of the research, the authors intended to study the variation of either Manning's roughness coefficient or Chezy's discharge coefficient as a function of the characteristics of the flow and properties of the sediment. It was found later that such an analytical approach is not likely to succeed. A new velocity formula was attempted,(1) the result of which is presented in this paper.

In order to understand the problem of determining the mean velocity of an alluvial stream more clearly, it is necessary to understand the mechanics of turbulent flow in pipes, in open channels and even in the turbulent boundary layer, and also to understand the mechanics of sediment transport. After considerable review of literature the authors came to the conclusion that a

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a. Presented at June 1958 Convention in Portland, Oregon.

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theoretical approach to the problem cannot be obtained at the present time, therefore an empirical approach was adopted. In order to facilitate the empirical correlation the method of dimensional analysis was used so that none of the most significant parameters would be omitted. A consistent empirical correlation was found and this has been further reduced to an exponential formula for practical application.

II. Literature Review

Most literature concerning the equations of estimating mean velocity or discharge is for clear flow in rigid conduits, either pipes or open channels. About 1768 Chezy[2] proposed a method of estimating the mean velocity of a stream by comparing the flow conditions with those of another having similar conditions. Such a proposition has been customarily written in a form known as Chezy's formula,

$$V = C\sqrt{RS} \tag{1}$$

in which V is the mean velocity, C the Chezy discharge coefficient, R the hydraulic radius, and S the slope of the channel.

In 1869 Ganguillet and Kutter[2] suggested a formula for determining Chezy's C:

$$C = \frac{a + \frac{b}{n} + \frac{m}{S}}{m + \left(a + \frac{m}{S}\right)\frac{n}{\sqrt{R}}} \tag{2}$$

in which a, b, and m are constants and n is a roughness factor.

In 1889, Manning[3] proposed several formulas for estimating the mean velocity of turbulent flow in conduits. The following well known Manning's formula was included in his original paper.

$$V = M R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{3}$$

in which M is an empirical constant depending upon the boundary roughness. However, Manning did not recommend its use because the equation is not dimensionally homogeneous. Eq. (3) is currently written for the English system as

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{4}$$

in which n is the Manning's roughness factor.

The authors introduce these commonly-used formulas here as reference showing the variation of the exponent of the hydraulic radius and that of the slope. Additional information regarding empirical velocity formulas can be found in the book "Hidraulik" written by Dr. S. Kolupaila.[4] Kolupaila shows that numerous exponents of the hydraulic radius and of the slope have been proposed in the past.

From the analytical point of view the mean velocity of a turbulent flow depends upon the velocity distribution, which is related to the mechanics of turbulent flow along boundaries. The equation of motion for turbulent flow is known as the Reynolds equation which differs from the common form of the Navier-Stokes equation by additional terms called the Reynolds stresses. T

olutions of the Reynolds equations will represent properly the turbulent flow. ecause the three Reynolds equations together with the equation of continuity or turbulent flow are not sufficient to determine the Reynolds stresses, ad-itional equations must be obtained either through hypothesis or through ex-erimental measurements to evaluate the unknowns.

Among the various formulas of velocity distribution proposed for turbulent ow, the logarithmic law (see Eq. (5)) is frequently used by hydraulic engi-eers. A brief review of this law may be helpful to understand its limitation f application. The logarithmic law (Eq. (5)) can be obtained either from randtl's hypotheses[5] of mixing length by assuming that, near the wall, the iixing length is linearly proportional to the distance from the wall and the iear stress is constant, or from Karman's similarity hypothesis[6] by as-uming that the mixing length is only a function of the velocity distribution nd the shear stress is constant. Therefore, this logarithmic law is for urbulent flow near rigid boundaries. It can be written as[7]

$$\frac{u}{V_*} = \frac{1}{\kappa} \ln\left(\frac{z}{z_0}\right) \tag{5}$$

n which u is the local mean velocity along the flow direction at a distance z rom the boundary, V_* is the shear velocity $\sqrt{\frac{\tau_0}{\rho}}$ in which τ_0 is the local oundary shear, κ is the so-called Karman universal constant and the value f z_0 is dependent upon a length parameter indicative of the hydraulic con-ition of the boundary.

From Nikuradse's data for turbulent flow in pipes, it can be found that in ase that $V_* k_s/\nu$ is less than about 3.5, in which k_s is the size of the sand sed in the experiments, the boundary can be classified as hydraulically mooth and Eq. (5) can be written for flow outside the laminar sublayer as[7]

$$\frac{u}{V_*} = \frac{2.3}{\kappa} \log_{10} \frac{V_* z}{\nu} + 5.5 \tag{6}$$

n case that $V_* k_s/\nu$ is greater than about 70, the boundary can be classified s hydraulically rough, and Eq. (5) can be written as[7]

$$\frac{u}{V_*} = \frac{2.3}{\kappa} \log_{10} \frac{z}{k_s} + 8.5 \tag{7}$$

ikuradse[8] found that the logarithmic law is not applicable to the flow near e center of the pipe, which is self evident according to the assumptions used n the derivation of the law. If the logarithmic law were exact to describe the elocity distribution of turbulent flow in pipes, the total discharge, and hence e mean velocity of the flow, could be determined by integration through the se of the logarithmic law. It was found that the constants in the resultant quations have to be modified in order to yield satisfactory results.

In general the formula of mean velocity for turbulent flow in a smooth pipe s[9]

$$\frac{V}{V_*} = C_1 \log_{10} \frac{V_* R}{\nu} + C_2 \tag{8}$$

nd that for turbulent flow in a rough pipe is[9]

$$\frac{V}{V_*} = C_1 \log_{10} \frac{R}{k_s} + C_3 \tag{9}$$

in which C_1, C_2, and C_3 are constants and R is the hydraulic radius.

Keulegan[9] applied Nikuradse's results to open channel flow. He showed that when the hydraulic radius is used as the characteristic length, the Nikuradse formula for pipe flow can be applied to open channel flow. However, Powell[10] found that because of the existence of a free surface in the open channel flow such an extension of Nikuradse's work to open channels ca not be done successfully. Additional information for flow in open channels composed of artificial roughness element on the boundary can be found from the works of Albertson and Robinson,[11] Sayre[12] and Johnson.[13]

In the foregoing review of the logarithmic law, there are two points whicl are important to the present study; (1) although the logarithmic law for turb lent flow near rigid boundaries has been verified by experimentation, the Karman-Prandtl hypotheses have not been proved to be theoretically sound, (2) the classification of the boundary roughness is in accordance with the co cept of the boundary layer.

Millikan[14] raised some doubts about the Karman-Prandtl hypotheses an showed that without employing these hypotheses the velocity distribution of turbulent flow in pipes or channels follows the logarithmic law in the overla zone where the "law of wall" and the "velocity-defect law" are both applicab The law of wall, which is due to Prandtl by use of a dimensional analysis, ca be written as:[14]

$$\frac{u}{V_*} = F_1\left(\frac{V_* z}{\nu}, \frac{z}{k_s}\right) \qquad (z \to 0) \qquad (1$$

The "velocity-defect law" is essentially empirical, first enunciated in its general form by Karman and can be written as[14]

$$\frac{U_{MAX.} - u}{V_*} = G\left(\frac{z}{h}\right) \qquad \left(\frac{z}{h} \not\to 0\right) \qquad (1$$

in which $u = U_{max}$ at $z = h$, and h is the value of z at the center of a pipe or two-dimensional channel.

Further discussion on velocity distribution will be presented later in con nection with the review of the turbulent boundary layer. The following remarks may be related to the classification of boundary roughness:

In the case of a rough boundary, the effect of viscosity on the velocity distribution can be neglected. A discharge formula, such as Manning's whic does not consider the effect of viscosity on the mean velocity, hence on the discharge, is only applicable to the case of turbulent flow along rough boundaries.

In the case of a smooth boundary, the effect of the roughness elements o the velocity distribution can be neglected. In addition to Eqs. (6) and (8), which resulted from the Karman-Prandtl hypothesis, there is another form known as the $\frac{1}{7}$ -power velocity-distribution law:[15]

$$\frac{u}{V_*} = 8.74\left(\frac{V_* z}{\nu}\right)^{\frac{1}{7}} \qquad (1$$

Eq. (12) was first discovered by Prandtl from the following Blasuis' empiri law of friction:[16]

$$f = \frac{0.316}{\left(\frac{VD}{\nu}\right)^{\frac{1}{4}}} \qquad (1$$

n which

$$f = 8\left(\frac{V_*}{V}\right)^2 \tag{14}$$

nd D_0 is the diameter of the pipe, and f is the Darcy-Weisbach resistance oefficient. According to Schlichting,(15) the exponents in Eqs. (12) and (13) re not constants as the Reynolds number increases, but dependent upon the eynolds number of the mean flow. Eq. (13) can be written as an exponential ype of discharge formula which will be discussed later.

Since the problem of the mean velocity and the velocity distribution of urbulent flow in open channels is essentially one of a turbulent boundary ayer, a brief review of the literature on the turbulent boundary layer along a lat plate at constant pressure may shed some light on the problem of velocity istribution in open channels.

From extensive wind tunnel measurements it is found that the mixing length heory has many limitations and inconsistencies. At the present time, scientists seem to be in favor of using statistical mechanics to study turbulence. completely satisfactory theory of turbulence is not available; scientists are eeking for laboratory data so that some satisfactory theory of turbulence can e formed.

It has been found(17) in the wind tunnel that the logarithmic law is valid nly within about 15 per cent of the thickness of the turbulent boundary layer. ccording to Clauser,(17) the flow within the turbulent boundary layer can be ivided into two regions. In the inner region, the law of wall is applicable; in he outer region, the velocity defect law is applicable. In the overlapping one where both the law of wall and the velocity-defect law are applicable, he logarithmic velocity distribution prevails, which is similar to Millikan's onclusion for turbulent flow in pipes and in channels.

Also according to Clauser the inner portion of the layer responds to the all shear much faster than the outer portion. While the inner portion completes its response within a few boundary layer thicknesses traveled, the uter portion takes a distance of tens or even hundreds times of the boundary ayer thickness for a corresponding response. A comparison of the response istance and mode of response to disturbances of various kinds and intensies confirms that a boundary layer is a truly non-linear phenomenon. Consequently, progress cannot be made by applying a linear concept of predeterminable response distances or times. Since the outer portion does not respond the wall shear very quickly, the velocity distribution in the outer portion epends also on the history of the flow. Although the law of wall has been ound to be independent of the pressure gradient along the boundary, it has not een proven to be applicable to the case where the boundary is movable or exible, such as the case of alluvial boundaries.

In brief it can be stated that, at present, there is no satisfactory theory of urbulent flow available so that the complete velocity distribution in turbulent ow can be calculated or predicted. Furthermore, since the flow is nonnear in nature, it is very doubtful that a theoretical and exact solution of the urbulent flow problem will soon be available, even though an approximate olution may be possible after extensive experimentation.

Another important factor in the study of the mean velocity in alluvial hannels is the presence of appreciable sediment transport which is absent rom flow in rigid channels. For flow transporting sediment, there are two ajor problems involved: (1) the amount of sediment transport, (2) the

problem of channel roughness and its effect on the discharge of the flow. There are several formulas ((18), (19) and (20)) for estimating the amount of sediment transported On the other hand, there is very little literature proposing velocity formula for alluvial streams. In case the bottom is plane, th alluvial boundary has been treated as a rigid one. For example, Strickler[21] proposed that the Manning's roughness factor can be expressed as a function of the sediment size for small gravels and cobbles:

$$n = 0.0160\ d^{\frac{1}{6}} \qquad d \text{ in mm.} \qquad (15a)$$

or

$$n = 0.039\ d^{\frac{1}{6}} \qquad d \text{ in ft.} \qquad (15b)$$

One of the major difficulties in determining the mean velocity of alluvial streams is that the bed configuration usually changes with the flow condition Consequently the bed roughness, which affects the velocity, changes with the flow condition. In 1950, Einstein and Barbarossa[22] proposed that the bound ry shear of a dune bed be divided into two portions: (a) that pertaining to the grain roughness and, (b) that pertaining to the dune roughness. Although suc an approach seems logical, its application to practical problems is still very limited.

Vanoni and Brooks[23] have shown that suspended load can cause a reduction in the resistance coefficient. They claim that the discharge and sedi ment load cannot be expressed as unique functions of the depth, slope and sa size. This point of view is shared by some investigators of the problems of flow transition due to the sudden change of the bed roughness.

In conclusion it can be stated that the theory of turbulent flow is still not complete even for flow near rigid boundaries. Its development for the case of a flow near a movable boundary seems even more remote. Moreover, the effect of sediment transport on the resistance coefficient is still unknown. Hence no theoretical analysis can be made at the present time, for the problem of mean velocity of alluvial streams. Therefore, an empirical corr lation seems to be desirable for engineering purposes.

III. Analysis of the Problem of Mean Velocity in Straight Alluvial Channels

In the case of flow carrying sediment, the change of flow causes not only the change of sediment transport but also the change of bed configuration. The phenomenon of sediment transport can be described by assuming that: (1) the bed material is granular and cohesionless, (2) the amount of supply o the sediment is equal to the amount of sediment transport, and (3) the flow is turbulent, steady and uniform. Let the bed be initially plane at a small discharge with no sediment moving, as the discharge increases, the following change of bed forms may be considered as being typical.

(a) Plane bed—Movement of sand grains occurs by rolling and sliding—occurring intermittently at random spots on the bed. As the discharge is further increased, the movement of sediment becomes more intense It can be stated that statistically there is a critical condition under which the movement of sediment begins.

(b) Ripples—As the discharge is increased still further ripples appear on the bed at a certain stage. A ripple bed is characterized by a rather regular wave pattern. The amplitudes of the ripples are usually small compared to their wave lengths. The characteristics of ripples are such that they eventually will become asymmetric, as demonstrated by Exner.(24)

(c) Dunes—At a later stage, dunes appear on the bed. A dune bed is usually characterized by a long upstream slope with a steep downstream slope. The sediment is eroded along the upstream slope and deposited in the trough. The pattern of sand dunes is not as regular as that of ripples. The change of bed surface from plane to ripples and dunes normally causes an abrupt change of bed roughness.

Bars—On further increase of the discharge the dune pattern is considerably modified. The dune-spacing is elongated, and the dune-crest is flattened. Some of the dunes are washed out. This bed form may be called bars. The hydraulic roughness of bars is not as high as that of dunes.

(d) Flat smooth bed—As the discharge is further increased a stage is reached at which the bed becomes approximately plane. The bed roughness at this state is less than that of dunes. This is a transition stage between dunes and antidunes. Vanoni and Brooks(23) call this type of bed form as flat bed. A flat bed is always formed by the hydraulic action of the flow, its deposition is generally very firm. For given bed material the hydraulic roughness of a flat bed may be smaller than that of a plane bed. Hence it is called flat smooth bed in this study. A flat bed is always formed by the hydraulic action of the flow, its deposition is generally very firm.

(e) Antidunes—In the antidune regime, sediment is deposited on the upstream slope of a sand wave and eroded from the downstream face of the sand wave, consequently the sand wave moves upstream while the sediment is transported downstream.

The various bed configurations can be estimated from Fig. 1(25) for a given ›w depth, slope, bed material size, and fluid temperature. Fig. 1 is copied ›m an article entitled "Discussion on Mechanics of Sediment Ripple For- ation"(25) by the first author. A thorough understanding of the development this figure is necessary for its application here in estimating the mean ve- city of flow. The two parameters $\frac{V_*}{w}$ and $\frac{wd}{\nu}$ in Fig. 1 are substitutes for - and $\frac{V_*d}{\nu}$ which were proposed originally.(27) This is permissible because $\frac{d}{} = \frac{V_*}{w} \cdot \frac{wd}{\nu}$. If the pair of parameters $\frac{V_*}{w}$ and $\frac{V_*d}{\nu}$ is used, the mechanics on which the criteria for various bed configuration are based can be under- ›od more clearly. However, the shear velocity V_* is contained in both the dinate $\frac{V_*}{w}$ and the abscissa $\frac{V_*d}{\nu}$, therefore a method of trial and error is re- ired in order to solve for it. On the other hand, if the pair of parameters and $\frac{wd}{\nu}$ is used as shown in Fig. 1, the mechanics upon which the criteria various bed configuration are based cannot be visualized easily, but, no al and error procedure is needed to solve for the boundary shear. In Fig. 1,

Shield's[26] criterion for the beginning of motion and Liu's[27] criterion for the beginning of ripples are shown respectively. A modified criterion for beginning of ripples and criteria for the formation of dunes and for transition proposed by Albertson, Simons and Richardson[28] are also shown. It should be pointed out that Fig. 1 is based mainly upon laboratory data for which the depth of flow is usually small. In order to maintain a laboratory flow with a bottom shear velocity comparable to that of a natural stream, the Froude number has to reach values considerably higher than that of natural streams. Furthermore, the relative grain roughness of a laboratory flow is generally greater than that of a large natural river. Such factors are not represented in Fig. 1. The authors also found that for lack of exact and unified definition of various bed configurations, data from various sources do not always agree with the classification shown in Fig. 1. Fortunately most data used by the authors in the analysis contain information on bed forms. In case the information on bed forms was missing, Fig. 1 was used as a guide in determining the bed form.

The change of the Darcy-Weisbach resistance coefficient f as a result of the change of bed configuration is illustrated by Fig. 2. The curves are for pipe flow after Nikuradse, and the points are data of No. 8 sand pertaining to the movable bed taken from Report No. 17 by the U. S. Waterways Experimental Station.[29] As long as the bed is plane, the variation of the resistance coefficient as the flow changes is similar to that of pipe flow. The sudden increase of the resistance coefficient indicated by the points occurs as sand waves appear on the bed. From this it is easy to see that application of the Nikuradse approach to the case of alluvial channels is not likely to be successful.

The exponential type of discharge formula is already in existance for flow in rigid channels. For example, the Blasius equation which is for turbulent flow along a smooth boundary can be written in exponential form as

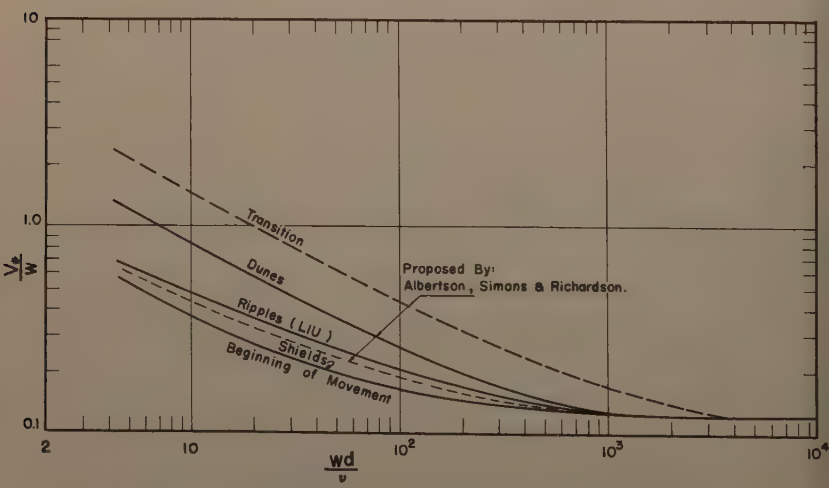

FIG. 1 CLASSIFICATION OF THE CONFIGURATION OF ALLUVIAL BED

$$V = \frac{56}{\nu^{\frac{1}{7}}} R^{\frac{5}{7}} S^{\frac{4}{7}} \tag{16}$$

hich for t = 65° F, ν (water) = 1.12×10^{-5} ft^2/sec. Therefore

$$V = 285 \ R^{0.714} S^{0.57} \tag{17}$$

ote that according to Eq. (16), a variation of temperature of 20° F from 65° F hanges the mean velocity about 4 per cent. Hence the effect of water emperature on mean velocity is normally negligible if the correct formula is sed. The Manning formula (Eq. (4)) which is for turbulent flow near a rough oundary is also an exponential formula. Note that these two formulas, Eqs.) and (16), are for extreme cases, and the exponents are not the same. It is ossible that the exponents of the discharge formula for turbulent flow in the ransition region have other values. In general the exponential type of dis- harge formula can be written as

$$V = C' \ R^{x} \ S^{y} \tag{18}$$

n which C' is an empirical coefficient, x and y are pure numbers. In lasius' formula x = 5/7 and y = 4/7, and in Manning's formula x = 2/3 and = 1/2.

For two-dimensional, steady, uniform flow, the depth of flow D can be con- idered dependent mainly upon the following variables: q the unit discharge f the flow, S the slope of the channel, ρ the fluid density, μ the fluid viscosity, the gravitational constant, d the 50 per cent size, i.e., the mean size of the ed material, σ the standard deviation of the size of the bed material, $\Delta\gamma_s$ the ifference in specific weight between the bed material and the fluid, and η the hape factor of the sediment, namely,

$$D = \phi_1 (q, S, \rho, \mu, g, d, \sigma, \Delta\gamma_s, \eta) \tag{19}$$

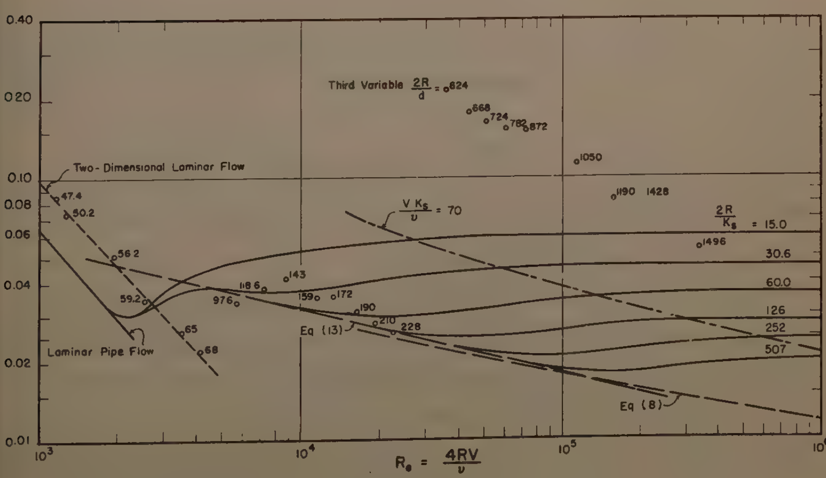

FIG. 2 VARIATION OF f WITH $\frac{4RV}{\nu}$ FOR ALLUVIAL BED

Since $q = DV$, and the fall velocity of the sediment particle w is a function o
d, ρ, μ, $\Delta\gamma_s$ and η, Eq. (19) can be written as

$$D = \phi_2(S, V, \rho, \mu, g, d, \sigma, \Delta\gamma_s, w) \qquad (2$$

By use of the π-theorem with D, V, and ρ as repeating variables, Eq. (20) c
be written as

$$\phi_3\left(S, \frac{V}{\sqrt{gD}}, \frac{VD}{\nu}, \frac{d}{D}, \frac{\sigma}{D}, \frac{V^2\rho}{\Delta\gamma_s D}, \frac{V}{w}\right) = 0 \qquad (2$$

If the effect of sediment mixture on the flow depth is considered to be secon
dary, the term σ/D can be omitted from the equation. It is difficult to use
Eq. (21) because there are four terms containing the unknown mean velocity
V. In order to avoid this difficulty the following transformation can be mad

$$\frac{V}{\sqrt{gD}} = \frac{V}{V_*}\sqrt{S} \qquad (22$$

$$\frac{VD}{\nu} = \frac{V}{V_*}\,\frac{V_* d}{\nu}\,\frac{D}{d} \qquad (22$$

$$\frac{\rho V^2}{\Delta\gamma_s D} = \frac{\rho V_*^2}{\Delta\gamma_s D}\left(\frac{V}{V_*}\right)^2\frac{d}{D} \qquad (22$$

$$\frac{V}{w} = \frac{V}{V_*}\,\frac{\frac{V_* d}{\nu}}{\frac{wd}{\nu}} \qquad (22$$

therefore a new set of dimensionless terms can be substituted in Eq. (21) su
as

$$\phi_4\left(\frac{V}{V_*}, S, \frac{V_* d}{\nu}, \frac{d}{D}, \frac{\rho V_*^2}{\Delta\gamma_s d}, \frac{wd}{\nu}\right) = 0 \qquad (23$$

In Eq. (23a), ρV_*^2 can be written as T_b the boundary shear at the bed level.
For a wide channel T_b is equal to γDS. For a narrow channel the wall-effe
may be appreciable. T_b then becomes $\gamma R_b S$ where R_b is the hydraulic radi
pertaining to the bed. The corresponding equation of Eq. (23a) for a narrow
channel is

$$\phi_5\left(\frac{V}{V_*}, S, \frac{V_* d}{\nu}, \frac{d}{R_b}, \frac{T_b}{\Delta\gamma_s d}, \frac{wd}{\nu}\right) = 0 \qquad (23$$

where $T_b = \rho V_*^2 = \gamma R_b S$. Notice that the two variables $\frac{V_* d}{\nu}$ and $\frac{wd}{\nu}$ can als
be written as $\frac{V_*}{w}$ and $\frac{V_* d}{\nu}$ or $\frac{V_*}{w}$ and $\frac{wd}{\nu}$.

In the case the grain of the bed material is spherical the value of $\frac{wd}{\nu}$ for
quartz sand moving in water depends upon only the grain size and the water
viscosity. The term $\frac{wd}{\nu}$ in Eq. (23) then can be omitted.(25)

In the following analysis, Eq. (23) is used as a guide in the correlation o
data.

IV. Empirical Correlation of Data

No specific laboratory work was done by the authors for this research. cisting data on water transporting natural sediment were collected as much possible. Both laboratory data and canal data were used. Although most the data were for flow having bed load only, considerable data for flow havg both suspended load and bed load were used. The depth of flow ranged om a few inches to several feet. The velocity of flow varied from less than ie foot per second to six feet per second. The slope of flow varied from 0004 to 0.028. The size of sediment varied from .01 mm to 80 mm (approxiately 3-1/2 inches). Both uniform bed material and graded bed material ere used. The variation of flow viscosity and sediment density of these data ere not appreciable. The effect of side wall was corrected according to the andard procedure.(30,35)

As explained earlier a theoretical treatment of the problem of mean vecity is not possible at the present time. The result of this study, therefore, s been obtained from empirical correlation based upon physical reasoning, mensional analysis, and the mechanics of boundary layer. The drawback of ing empirical correlation is that usually the parameters cannot be explained ther theoretically or physically.

To correlate the data the authors used three parameters $V_* d/\nu$, $\frac{wd}{\nu}$ and

$$\frac{\frac{V}{V_*}\ \frac{\tau_b}{\Delta\gamma_s d}\ S^{\lambda}}{\left(\frac{R_b}{d}\right)^{m} F_r^{N}}$$

ie term F_r is defined as $\frac{V}{\sqrt{gD}}$ for two-dimensional flow, and as $\frac{V}{\sqrt{gR_b}}$ r flow in a narrow channel where the wall-effect is appreciable. All the rameters, with the exception of F_r, are included in Eq. (23b). Note that the rameter F_r can be written in terms of $\frac{V}{V_*}$ and S according to Eq. (22a). ierefore, only those dimensionless parameters of Eq. (23b) are used.

There remains a question whether the flow depth D in $\frac{V}{\sqrt{gD}}$ can be reac d by R_b. The ratio $\frac{V}{\sqrt{gD}}$ $\left(\text{or } \frac{V^2}{gD}\right)$ is known as either the Froude number the Kineticity of flow. It is defined as either the flow velocity divided by e celerity of the gravitational wave, or twice the kinetic energy divided by e potential energy of the flow. Obviously both definitions cannot be applied the parameter $\frac{V}{\sqrt{gR_b}}$. However, the substitution of R_b for D in Eq. (22a) permissible, provided that the term $\frac{V}{\sqrt{gD}}$ is used in connection with the w resistance. Then $\frac{V}{\sqrt{gD}}$ can be considered as index for the energy loss used by surface waves, which influence the mean velocity V, the boundary ear V_* and the energy gradient S. Therefore the parameter $\frac{V}{\sqrt{gR_b}}$ or its uivalent $\frac{V}{V_*}\sqrt{S}$ can be considered as an index indicating the energy loss used by the surface waves.

The first parameter is the shear-velocity Reynolds number and the secon parameter is the fall-velocity Reynolds number of the grain. The third parameter, which can be abbreviated as K, i.e.,

$$K = \frac{\frac{V}{V_*}\frac{\tau_b}{\Delta\gamma_s d}S^{\lambda}}{\left(\frac{R_b}{d}\right)^m F_r^N}$$

may need some explanation. These dimensionless parameters were evolved from a plot made by the first author(31) in his previous study of the roughne of alluvial beds. In this earlier study only $\frac{V_* d}{\nu}$, $\frac{V}{V_*}$ $\frac{T_b}{\Delta\gamma s d}$, $\frac{wd}{\nu}$ were used. The term $\frac{V}{V_*}$ $\frac{T_b}{\Delta\gamma_s d}$ was interpreted as the tractive force divided by the submerged weight of the particle multiplied by a resistance coefficient V_*/V.

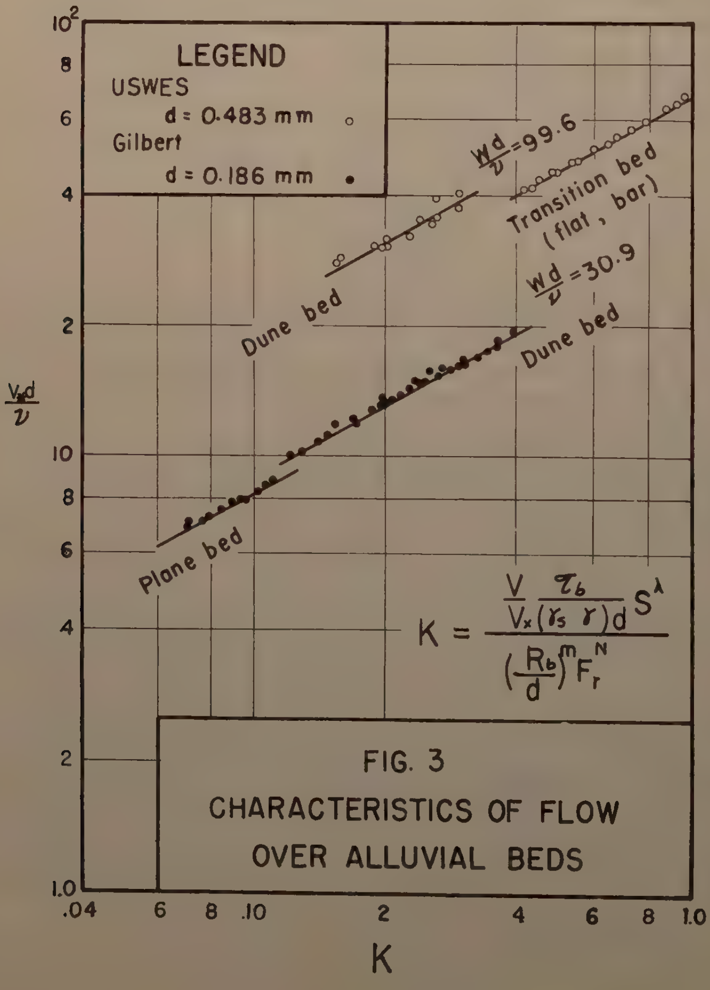

FIG. 3

CHARACTERISTICS OF FLOW OVER ALLUVIAL BEDS

addition to these parameters $\frac{R_b}{d}$, S and F_r were added empirically to the -term for the following reasons:

1. To conform with the existing knowledge of boundary resistance,
2. To correlate the data consistently.

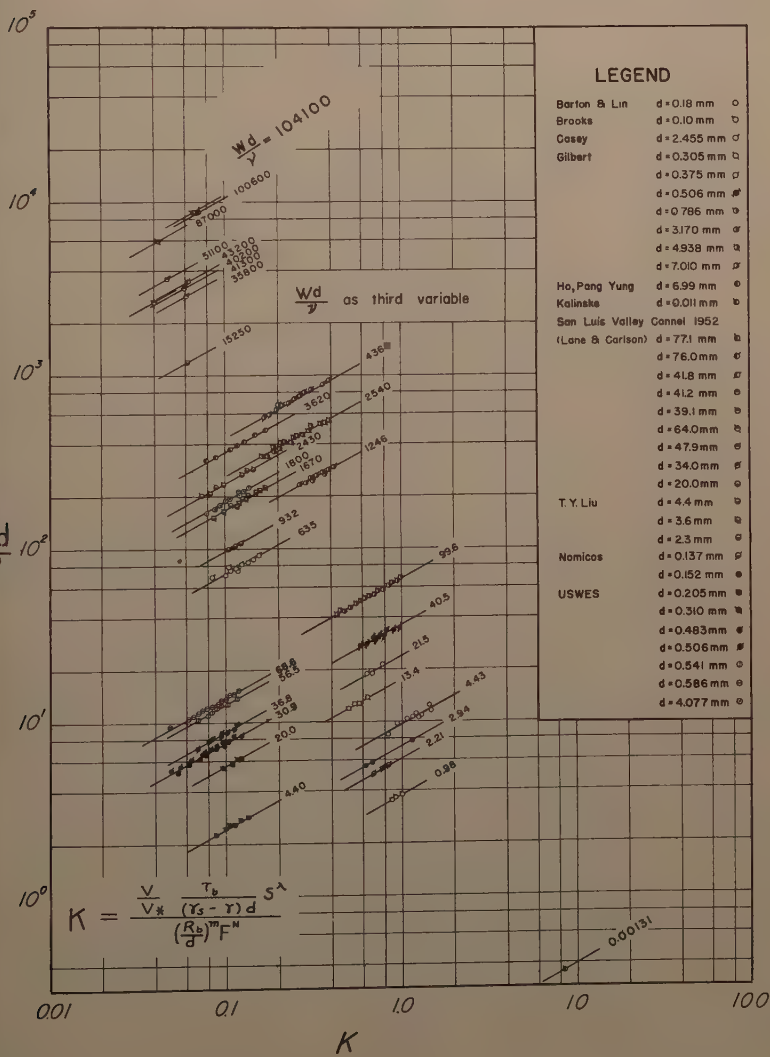

FIG. 4 CHARACTERISTICS OF FLOW OVER ALLUVIAL PLANE BED

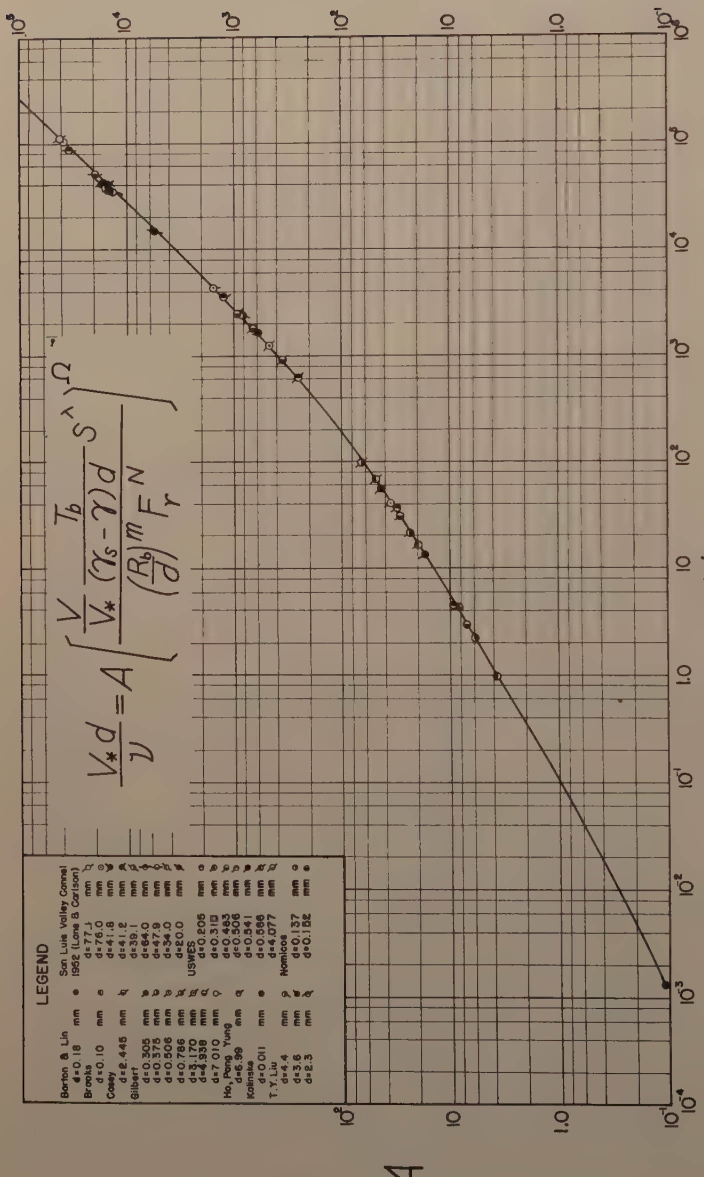

FIG. 5 VARIATION OF A WITH $\frac{Wd}{\nu}$ FOR ALLUVIAL PLANE BED

For any given constant value of wd/ν, the data, when plotted according to V_*d/ν against K shown in Fig. 3, were found to fall on two straight lines depending upon whether the bed is a plane bed or a dune bed. The condition at which the plane bed changes into wavy bed can be estimated according to Fig. 1 (ripples are considered as incipient dunes). The intercept of these straight lines with the line of K = 1 depends upon the third variable wd/ν. For clarity, straight lines pertaining to the plane bed have been plotted separately from those pertaining to the dune bed. These straight lines for a plane bed are shown in Fig. 4. Their slope is 1:0.555 horizontal to vertical, and the

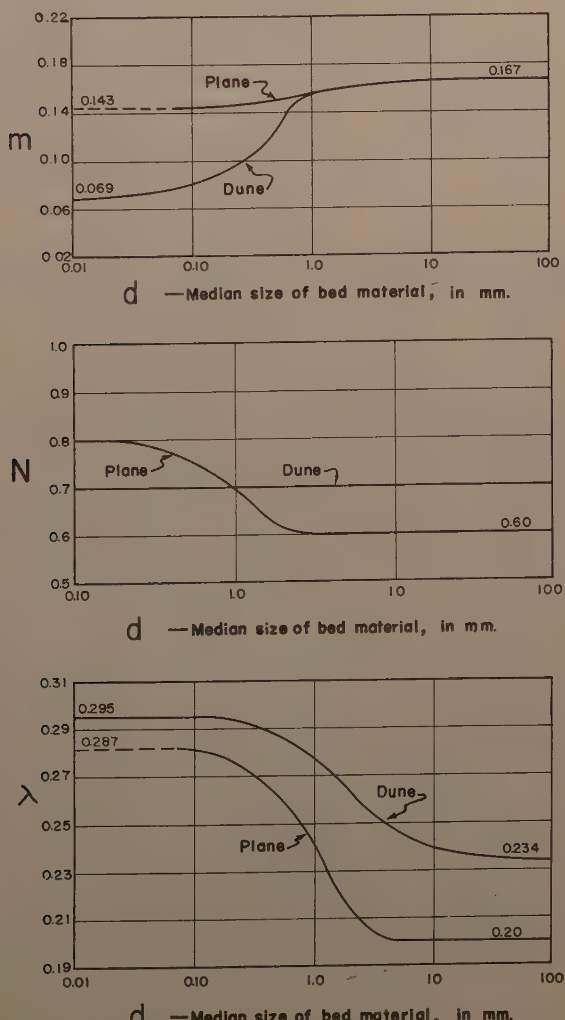

FIG. 6 VARIATION OF λ, m, AND N WITH d FOR FLOW OVER ALLUVIAL BED

intercepts with K = 1 are shown in Fig. 5 according to the third variable wd/ν which varies from 0.00131 to 104.100. A general equation can be written for the data for the plane bed

$$\frac{V_* d}{\nu} = A \left(\frac{\frac{V}{V_*} \; \frac{T_b}{\Delta\gamma_s d} \; S^{\lambda}}{\left(\frac{R_b}{d}\right)^m \; F_r^N} \right)^{\Omega} \qquad (24)$$

in which Ω is equal to 0.555, A is a function of the third variable $\frac{wd}{\nu}$ as shown in Fig. 5, and λ, m and N are pure numbers taken from Fig. 6 which were obtained empirically as functions of the mean size of the bed material. It should be mentioned that the curves for λ, m, and N have been chosen empirically so that data can be plotted on parallel straight lines shown on Figs. 4 and 7, in other words, if the values of λ, m, n, are to be plotted on Fig. 6, they should fall on the curves.

From Fig. 5, for $wd/\nu > 1000$, the factor A can be expressed as

$$A = \epsilon \frac{wd}{\nu} \qquad (25)$$

in which ϵ has an approximate value of 0.39, and is dependent upon the shape factor and also the fall-velocity Reynolds number of the grain of the bed material, and

$$w = \sqrt{\frac{4}{3} \frac{1}{C_D} \frac{\rho_s - \rho}{\rho} g d}$$

in which C_D is the drag coefficient of the grain of the bed material.

As shown in Figs. 5 and 8, the coefficient A in Eq. (24) depends upon the parameter $\frac{wd}{\nu}$ which was computed on the basis of spherical grains. In the previous discussion of Eq. (23b), it was mentioned that if the grain of the sediment is assumed to be spherical, the parameter $\frac{wd}{\nu}$ is not an independent variable and can be omitted. The inclusion of $\frac{wd}{\nu}$ in the data analysis seems to contradict the above statement. This contradiction actually does not exist for the following reasons:

(a) the variables needed for computing $\frac{wd}{\nu}$ namely d, $\Delta\gamma_s$, and ν are also used in computing other parameters in Eq. (24),

(b) the parameter $\frac{wd}{\nu}$ can be written as a combination of $\frac{V_* d}{\nu}$ and $\frac{T_b}{\Delta\gamma_s d}$ since $\frac{wd}{\nu}\sqrt{C_D} = \frac{2}{3}\frac{V_* d}{\nu} \Big/ \sqrt{\frac{T_b}{\Delta\gamma_s d}}$.

Furthermore the inclusion of $\frac{wd}{\nu}$ may make Eq. (24) useful even when the grain shape is considered. When Eq. (25) is substituted in Eq. (24), the result is:

$$V = 3.35 \left(\frac{C_D}{\epsilon^2}\right)^{\frac{4.5}{2}} d^{\frac{2\Omega(1-m)-1}{2\Omega(1-N)}} R_b^{\frac{1-\Omega(1+N-2m)}{2\Omega(1-N)}} S^{\frac{1-\Omega(1+2\lambda)}{2\Omega(1-N)}} \qquad (2$$

t high Reynolds number C_D is a constant, therefore, Eq. (26) does not conain the factor of viscosity, μ or ν. Eq. (26) is for turbulent flow near rough oundaries. If the values of λ, m, N, and Ω for plane bed of very large bed aterial, that is, $\Omega = 0.555$, $m = 1/6$, $N = 0.6$ and $\lambda = 0.2$ are substituted in q. (26), the result will be

$$V = 3.35 \left(\frac{C_D}{\varepsilon^2}\right)^{\frac{4.5}{2}} d^{-\frac{1}{6}} R_b^{\frac{2}{3}} S^{\frac{1}{2}} \tag{27}$$

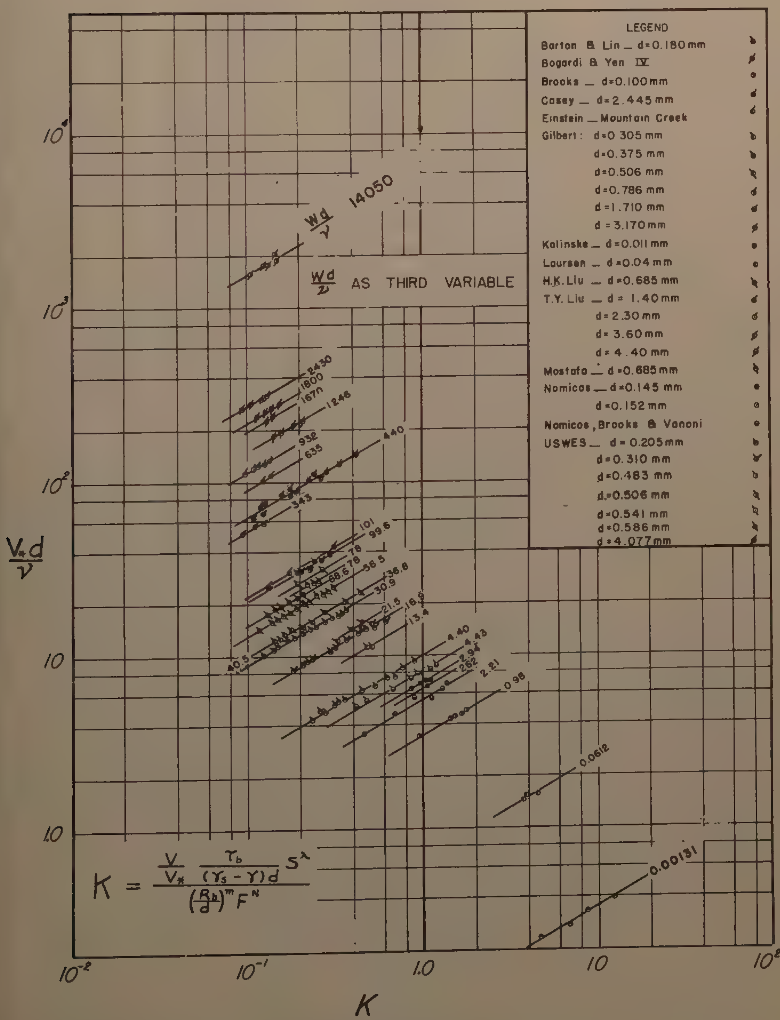

FIG. 7 CHARACTERISTICS OF FLOW OVER ALLUVIAL DUNE BED

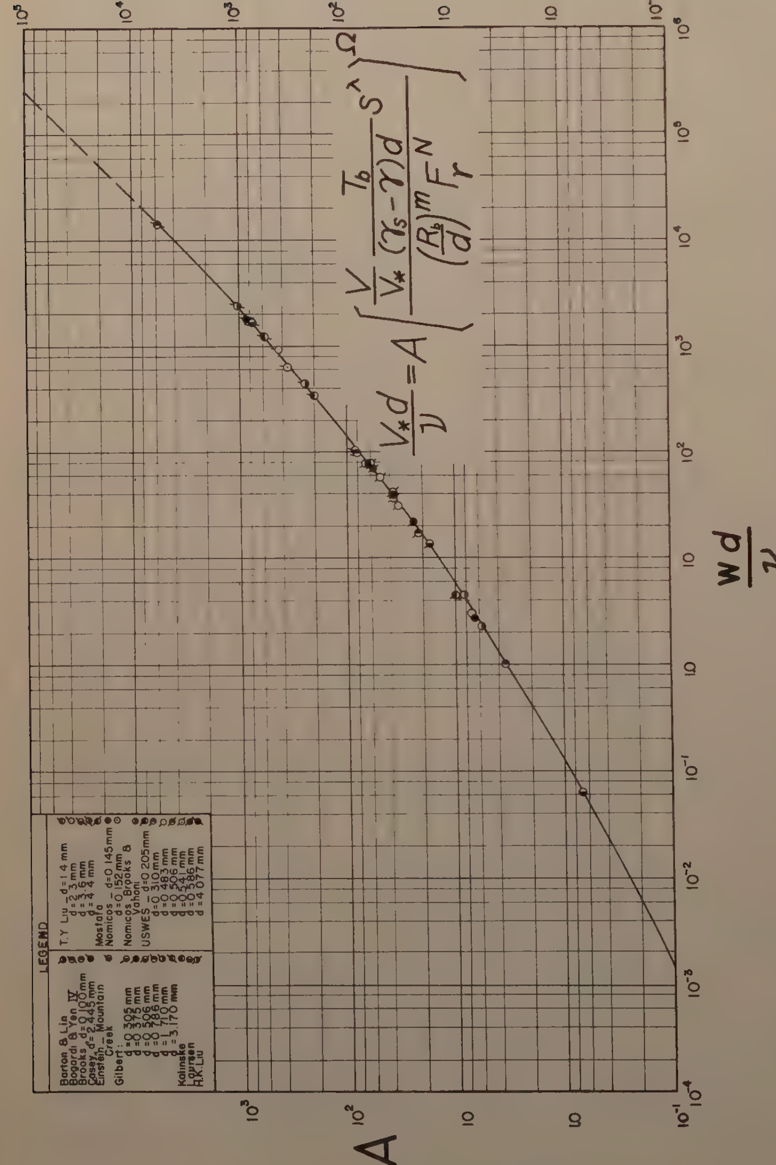

FIG. 8 VARIATION OF A WITH $\frac{Wd}{\nu}$ FOR ALLUVIAL DUNE BED

The exponents of d, R_b and S are the same as those given by the Manning-Strickler formula. If the drag coefficient C_D is chosen as 0.49, together with $\epsilon = 0.39$, Eq. (27) will become the ordinary Manning-Strickler formula. Theo-retically both the coefficient ϵ and the drag coefficient C_D are dependent upon he shape and the fall-velocity Reynolds number of the grain of the bed ma-erial. The fact that both ϵ and C_D are of the same order of magnitude and he fact that the exponent of ϵ is twice as large as that of C_D may explain why he mean velocity may be proportional to $C_D^{2 \cdot 25}$. The factors ϵ and C_D in Eq. (26) can be used to explain why the Strickler's coefficient is different rom that of Chang's,(32) which is

$$n = 0.0166\ d^{\frac{1}{6}} \qquad d \text{ in mm} \tag{28}$$

From Fig. 5, for $\frac{wd}{\nu} < 1$.i.e., within the range of Stokes' law the factor A can e written as

$$A = \theta \left(\frac{wd}{\nu} \right)^p \tag{29}$$

n which θ is a constant, approximately of 3.4, p is the slope of the curve, ap-roximately to be 1/2 and $w = 1/18 \frac{\rho_s - \rho}{\rho} g d^2$ for spherical grains. Substitut-ng Eq. (29) in Eq. (24) with $\theta = 3.4$ yields

$$V = 56\ \nu^{\frac{2p-1}{\Omega(1-N)}}\ d^{\frac{1+\Omega(1-m)-3p}{\Omega(1-N)}}\ R_b^{\frac{1-\Omega(1+N-2m)}{2\Omega(1-N)}}\ S^{\frac{1-\Omega(1+2\lambda)}{2\Omega(1-N)}} \tag{30}$$

For the limiting values of λ, N, m and Ω for plane bed composed of very small ed material, that is, $\Omega = 0.555$, $N = 0.8$, $m = 1/7$ and $\lambda = 0.287$, as used, the xponent of R_b reduces to 0.72, the exponent of S to 0.57, the exponent of d educes to zero at $p = 0.492$, and consequently the exponent of ν reduces to /7. Eq. (30) is then reduced to the Blasuis equation (Eq. (16)) for turbulent low near smooth boundaries. Should the value of p be 1/2, the exponent of ν s then zero, which agrees with previous discussion that the effect of viscosity n the mean velocity is very small and can be neglected.

A similar correlation can be found for dune bed as shown in Figs. 7 and 8. he data shown on Figs. 7 and 8 can be represented also by Eq. (24), except hat Ω for dune bed is 0.565, and the exponents λ, m and N for dune bed are s shown also in Fig. 6.

Eq. (24) is considered to be the general equation representing the flow haracteristics of alluvial streams. Although Eq. (24) is dimensionally omogeneous, it is not convenient to use. A further simplification of Eq. (24) s discussed in the next chapter.

V. Proposed Discharge Formula for Alluvial Streams

For simplification Eq. (24) can be reduced to

$$V = C_a\ R_b^x\ S^y \tag{31}$$

which C_a is a discharge coefficient for alluvial streams and can be omputed by Eq. (32)

$$C_a = \Psi \, d^{\frac{1+\Omega(1-m)}{\Omega(1-N)}} \qquad (32$$

in which

$$\Psi = \left(\frac{P}{\Omega\sqrt{A}} \right)^{\frac{1}{1-N}} \qquad (32$$

in which P is not the p used in Eq. (29), but is

$$P = \frac{g^{\frac{1+\Omega(1-N)}{2\Omega}}}{\Omega\sqrt{\nu}} \; \frac{\rho_s - \rho}{\rho} \qquad (32$$

and

$$A = f\left(\frac{wd}{\nu}\right) \qquad (32$$

it can be seen from Eq. (32) that the discharge coefficient C_a is a function o d, ρ_s, ρ, g, and ν, and Ω, which depends upon the bed configuration. Furthe more, the bed material is generally composed of a mixture of non-spherica grains and the coefficient C_a depends also upon the shape factor and the standard deviation of the bed material. In Eq. (31) x and y are pure number and can be computed from Eqs. (33) and (34) respectively,

$$x = \frac{1-\Omega(1+N-2m)}{2\Omega(1-N)} \qquad (3$$

and

$$y = \frac{1-\Omega(1+2\lambda)}{2\Omega(1-N)} \qquad (3$$

in which λ, m and N are shown in Fig. 6 and

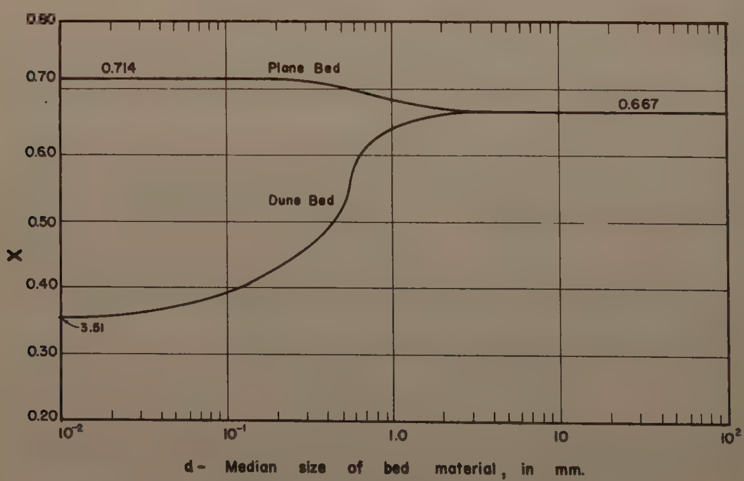

FIG. 9 VARIATION OF X WITH d FOR FLOW OVER ALLUVIAL BED

$\Omega = 0.555$ for plane bed
$\Omega = 0.565$ for dune bed

When the values of Ω, λ, m and N are substituted according to the bed configuration (plane bed or dune bed), the results of the exponents x and y are shown in Figs. 9 and 10 respectively as functions of the bed configuration and the size of the bed material. Because λ, m, and N are empirical, the curves for x and y are also empirical. Since x and y are pure numbers, it is reasonable to assume that they depend upon some dimensionless parameter, rather than upon the size of the bed material alone. The dimensionless parameter which is still unknown should be directly related to the boundary conditions, and/or to the flow conditions. The unknown dimensionless parameter may be some combination of those given in Eq. (23), with the possible exception of V/V_*. Such a dimensionless parameter has not been attempted by the authors. On the other hand, since the choice of x and y depends partly upon bed configuration which is governed by two dimensionless parameters, V_*/w, wd/ν and the later can be obtained by dividing $\frac{V_* d}{\nu}$ by $\frac{wd}{\nu}$, the effect of the hydraulic boundary condition on the choice of x and y has been partly, if not entirely, considered. Further research is needed to express the exponents x and y as functions of certain dimensionless parameters.

Note that in Fig. 9, the variation of the exponent x against the bed material size d for dune bed is opposite to that for plane bed. Both the exponent x for plane bed and that for dune bed are 2/3 when the bed material size d is greater than 4 mm (the exponent x of 2/3 is the same as appeared in Manning's formula.) For plane bed, x increases as d decreases for d smaller than 4 mm. The x-value reaches an upper limit of 5/7 as d becomes less than 0.2 mm (the exponent x of 5/7 is the same as appeared in the Blasius formula). For dune bed, x decreases as d decreases. The value of x is 0.35 when the value of d is 0.01 mm.

Note that in Fig. 10, the variation of the exponent y against the sediment size d for dune bed is also opposite to that for plane bed. The exponent y for

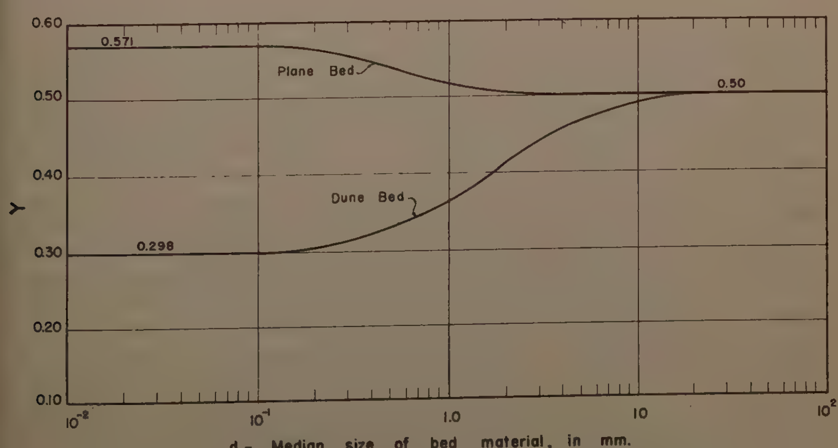

FIG. 10 VARIATION OF Y WITH d FOR FLOW OVER ALLUVIAL BED

plane bed is 1/2 when the bed material size is greater than 4 mm (the y-valu of 1/2 is the same as appeared in Manning's formula). As the size of the be material decreases, the exponent y for plane bed increases for d smaller tha 4 mm. The y-value reaches an upper limit of 0.57 when d becomes 0.1 mm or smaller (the y-value of 0.57 is the same as appeared in the Blasius formu la). For dune bed the exponent y becomes 1/2 when the bed material size is 20 mm or greater. For d smaller than 20 mm, y decreases as d decreases. The y-value is 0.30 when the d-value is 0.1 mm or smaller.

That the exponent y for dune bed is normally below 1/2 may need some di cussion: If

$$V \propto S^{\frac{1+\delta}{2}} \tag{35a}$$

it means that

$$S \propto V^{\frac{2}{1+\delta}} \tag{35b}$$

For turbulent flow near rough boundaries, $\delta = 0$ therefore S is proportional to V^2. In case of δ is greater than zero, $S \propto V^{\beta} < 2$, such as in the case of the Blasius equation. On the other hand, if δ is less than zero, it means $S \propto V^{\beta} > 2$, in other words, the head loss of the flow is proportional to the velocity with an exponent which is greater than 2. Note that according to Lacey's regime theory,(33) the mean velocity in a regime channel is proportional to the energy gradient to the one-third power:

$$V = 16.0 \; R^{\frac{2}{3}} \; S^{\frac{1}{3}} \tag{3}$$

By examination of Fig. 9 and Fig. 10, it can be concluded: (1) the Mannin formula is applicable for plane bed when the size of the bed material is 4 m or greater, (2) the Blasius formula is applicable for plane bed when the size of the bed material is 0.1 mm or smaller, (3) when the size of the bed mate al is 20 mm or greater the Manning formula is applicable regardless of the bed configuration, because the effect of dune formation, if any, on the mean velocity is negligible and (4) the formation of dunes generates additional energy loss so that the energy loss is proportional to the velocity with an exponent normally greater than 2.

The coefficient C_a can be computed from Eq. (32). However, such a method of determining C_a is very tedious. Instead the average C_a-curves were determined by substituting available data in Eq. (31) through the use o Figs. 9 and 10 for choosing x and y. Since C_a is not dimensionless, and its dimension depends upon the exponent of the hydraulic radius, therefore the C_a-value for the English system (Fig. 11) differs from that for the metric system (Fig. 12). The coefficient C_a should be a function of the properties the sediment and the fluid. Under ordinary conditions of sand transported b water flowing in open channels, the density of the sediment is approximatel constant. The shape of the sediment particle and the properties of water ca also be considered approximately constant. Therefore, C_a is essentially a function of the sediment size alone. It should be noted that the temperature variation of the data was between 15° C and 30° C (59° F and 86° F respectiv ly). It was found that the variation of C_a due to temperature change is less than that due to error of measurements. That the effect of the variation of viscosity of the water on the mean velocity is small has been shown to be tr in the Blasius formula. Therefore, the effect of the temperature on the

scharge coefficient can be neglected for practical purposes, provided that
e appropriate exponents for R_b and S are used.

Note that the effect of temperature is included in Eq. (24), which has been duced to Eq. (31). However, the effect of temperature is neglected in the termination of C_a-curves. The explanation for this may be that, if the scosity factor is used to form a certain dimensionless parameter in the ıdy of alluvial roughness, the inclusion of the effect of viscosity should be mplete. On the other hand, for practical purposes, the effect of neglecting scosity on the mean velocity may be small compared to errors of other urces.

Because the exponents for the hydraulic radius and the slope S depends on the bed configuration, separate C_a-curves for plane bed and dune bed are so necessary as shown both in Fig. 11 and in Fig. 12.

In the case of a plane bed, the discharge coefficient C_a decreases as the d material size d increases except when d is smaller than 0.1 mm, then the $_a$-value is essentially a constant equal to 287 (see Fig. 11). This is the case ıere the Blasius equation is applicable, and where the discharge coefficient independent of the height of the boundary roughness. Between d = 0.2 mm d d = 1 mm, the value of C_a decreases rapidly as the value of d increases. etween d = 4 mm and d = 80 mm, the value of C_a can be written as,

$$\text{For English system} \quad C = 112 - 30.5 \log_{10} d \quad (d \text{ in mm.}) \tag{37a}$$

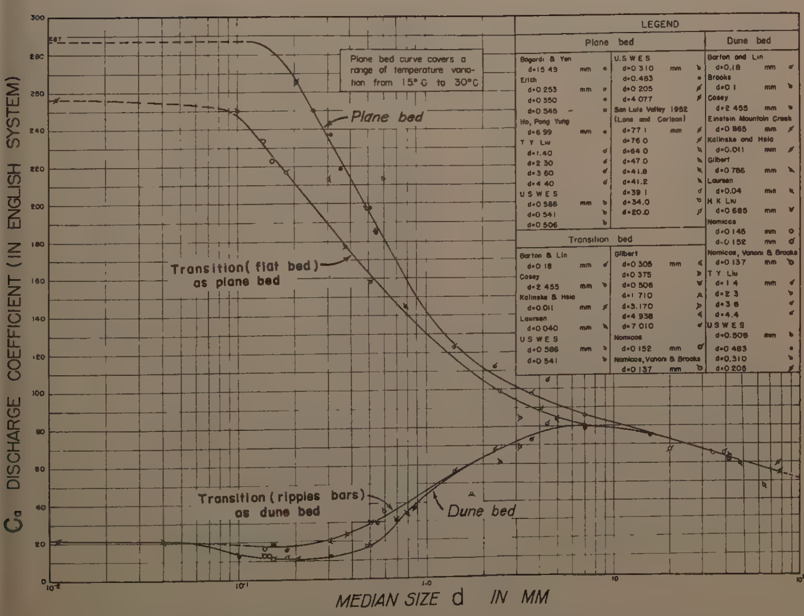

FIG. 11 VARIATION OF C_a (IN ENGLISH SYSTEM) WITH d FOR FLOW OVER ALLUVIAL BED

For Metric system $C = 75 - 21 \log_{10} d$ (3
(d in mm.)

Eq. (37) corresponds to the Strickler formula shown as Eq. (16). Notice th the drag coefficient C_D of a sphere decreases abruptly at about $\frac{wd}{\nu} = 2 \times 10$ which corresponds approximately to d = 80 mm at 20° C. Therefore Eq. (3 and Figs. 11 and 12 should be applied with caution for d larger than 80 mm

In the case of a dune bed, at the bed material size d equal to about 0.2 m the discharge coefficient is a minimum, which may be interpreted as that t effect of dunes on the discharge coefficient is the greatest at about d = 0.2 mm; and at the size of less than 0.04 mm, the discharge coefficient is essentially constant at 21. The discharge coefficient C_a increases as the siz of the bed material increases from .2 mm to 7 mm, which means that withi this range of the bed material size the effect of dunes on the discharge coe ficient decreases although the size of the bed material increases. The C_a- curve for dune bed coincides with that for plane bed at d = 20 mm, which means that for d equal to 20 mm or greater the effect of dune on the discha coefficient is negligible. The last statement agrees with the previous conclusion that the Manning formula is applicable for d larger than 20 mm regardless the bed configuration.

It was pointed out earlier that the bed configuration after the beginning motion can be classified as plane, ripples, dunes, bars, flat, and antidunes The case of antidune is excluded entirely from this discussion because of insufficient data. The most important concept out of this research is that flow over an alluvial bed is divided into two classes: that is, the flow with plane bed, and the flow with a dune bed. The discharge coefficient C_a and exponents x and y depend upon not only the bed material size but also the b configuration, plane bed or dune bed.

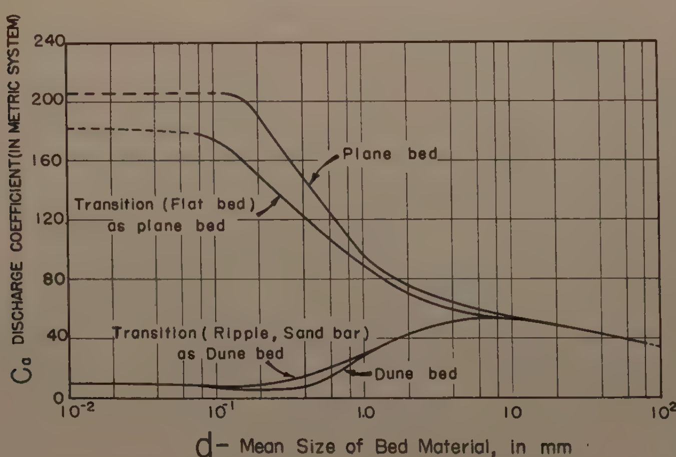

FIG. 12 VARIATION OF C_a (IN METRIC SYSTEM) WITH d FOR FLOW OVER ALLUVIAL BED

In the case of ripple bed, which is considered as the transitional stage be- veen the plane bed and the dune bed and in the case of sand bars, which llows the stage of the dunes, the values of the exponents x and y for the dune ed have been used in determining the C_a-value. The C_a-curve for ripples id sand bars is shown immediately above the C_a-curve for dunes. The two irves coincide with each other for the bed material sizes less than about 06 mm, and greater than 1.6 mm, which means the distinction between pples, bars, and dunes as far as their effect on the discharge is concerned nil for $d < 0.06$ mm and for $d > 1.6$ mm (It can be estimated from Fig. 1 at ripples will not form when d is greater than about 2 mm). That the C_a- irve for ripples and bars lies above that for dunes means that the resistance ripples or sand bars is generally less than that of dunes. The minimum $_a$-value for ripples and bars is about 18 at d equal to about 0.16 mm.

In the case of a flat bed, which follows the stage of sand bars, the values the exponents x and y for plane bed have been used in determining the C_a- ilue which is shown immediately below the C_a-curve for plane bed. The C_a- ilue reaches essentially a constant of 256 (see Fig. 11) when the bed materi- size is less than 0.1 mm. The C_a-curve of flat bed coincides with that of ane bed when the bed material size is equal to or greater than 20 mm. That e C_a-curve of flat bed in general lies below that of plane bed indicates that e discharge coefficient of a flat bed is normally less than that of a plane ed. This means, for given bed material, depth of flow and slope of the iannel, the discharge or velocity of a flow with flat bed is less than that of a ow with plane bed.

It should be noted that in order to select the exponents x and y and the ap- ropriate discharge coefficient C_a, it is necessary to estimate the bed configu- ation, which requires the use of Fig. 1. The following is an illustration to stimate the mean velocity of flow: Given from Run No. 32. Test No. 5 - 0010: U.S.W.E.S. Report No. 17,(29)

$$d = 0.483 \text{ mm}$$

$$S = 0.001$$

$$R_b = 0.397 \text{ ft}$$

$$t = 60.5^{\circ} \text{ F}$$

equired: V

omputation procedure:

$$V_* = gR_bS = 0.113 \text{ fps}$$

ssume the sediment is spherical, $w = 0.234$ fps and $\frac{wd}{\nu} = 30.9$ therefore

*= 0.483. From Fig. 1 it is estimated that the bed would be a dune-bed.

ierefore, the exponents and coefficient for a dune-bed at d = 0.483 are iosen:

From Fig. 11	$C_a = 17$
From Fig. 9	$x = 0.507$
From Fig. 10	$y = 0.328$

ibstitute these values in Eq. (31).

$$V = 17 \times 0.397^{0.507} \times 0.001^{0.328} = 1.11 \text{ fpS.}$$

The measured velocity was 1.19 ft per second.

The data computed in this manner are shown in Fig. 13 and Fig. 14. In Fig. 13 which is for a plane bed, 83 per cent of the data are within 10 per c of scatter. None of the computed velocity exceeds 20 per cent of deviation from the measured value. In Fig. 14, which is for dune bed, 70 per cent of the data are within 10 per cent of scatter and only 1 per cent of the data ex ceeds 20 per cent of deviation from the measured value. In general the av age error is about 10 per cent or less. In view of the fact that it is very d cult to obtain accurate data of flow in alluvial channels, such as measuring the depth of flow and the energy slope, an average discrepancy of 10 per c in computing the velocity can be considered acceptable for engineering purposes.

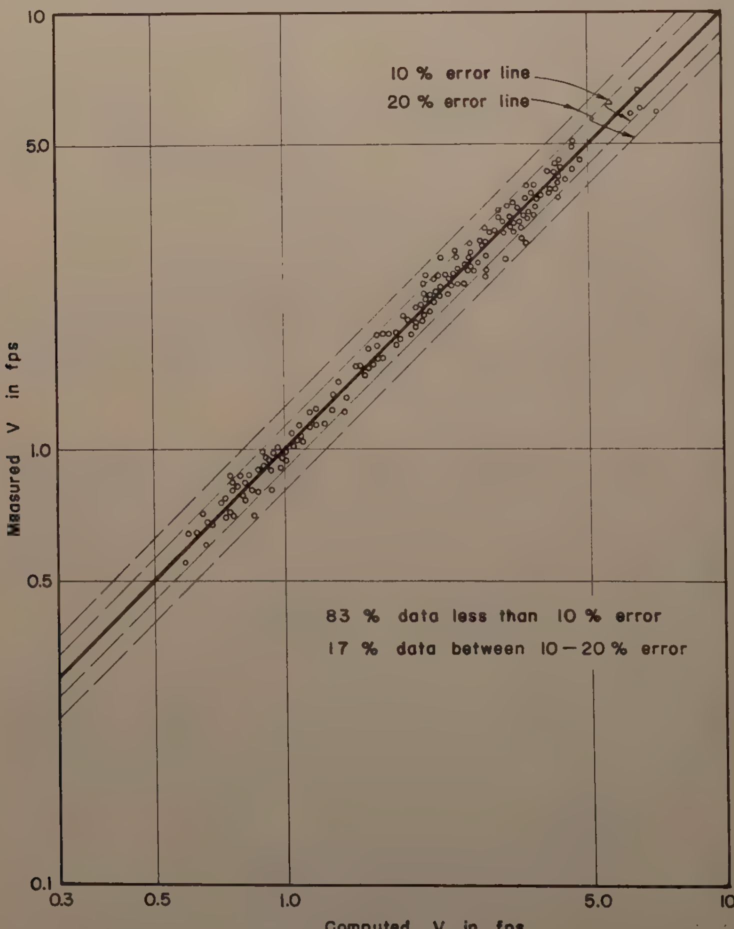

Fig. 13 A comparison of computed mean velocity with measured mean velocity for alluvial plane bed

The results presented here are primarily for flow in straight channels. e effect of a side wall should be eliminated by using standard pro- dures.(30,35) This involves a method of trial and error since R_b cannot be mputed without knowing the mean velocity. In order to use this method, th the total discharge and the depth of flow must first be assumed. The ean velocity can be found according to the equation of continuity, then R_b n be computed. By using R_b, the slope of the channel, and the mean size of e bed-material, the mean velocity can be checked according to Eq. (31). is method should be used through repeated trials until the result is satis- ctory.

Eq. (31) is suitable for a steady, uniform flow. However, this is not the se for most of the natural streams. In the case of natural streams the dis- arge coefficient probably has to be modified to suit the field condition. The

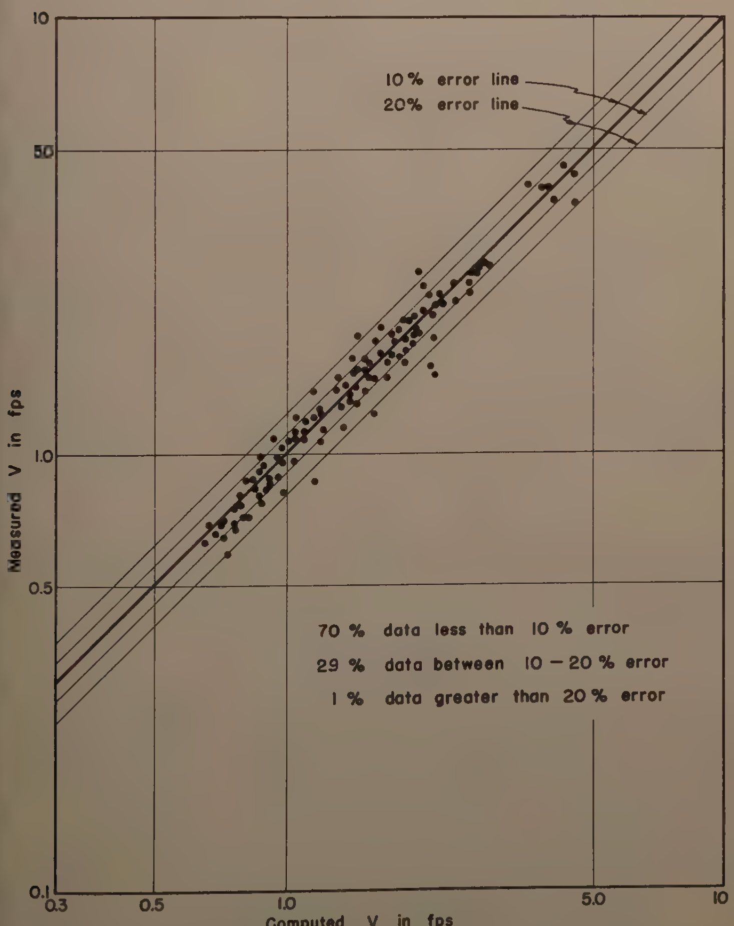

Fig. 14 A comparison of computed mean velocity with measured mean velocity for alluvial dune bed

exponents for hydraulic radius and slope probably can be the same as shown in Figs. 9 and 10—at least as a first approximation.

VI. Suggestions for Future Research

As pointed out earlier, it is impossible at the present to find a theoretic solution of the mean velocity of alluvial streams, therefore the authors have proposed certain methods of empirical correlation. In so doing it was nece sary to make some assumptions for simplification. However, in order to understand the problem thoroughly so that the final solution of mean velocit of alluvial streams can be obtained, some additional research work definite is needed. The needed research is almost unlimited. The following suggestions are only those which are directly related to the present approach.

1. The information on bed configuration is very important in order to ap ply the discharge formula properly, the classification of bed configuration needs to be defined more accurately.
2. In this study the effect of sediment shape was not considered. All se ment particles were assumed to be spherical. In order to improve th accuracy of the method, it is necessary to determine the effect of pa cle shape on the mean velocity.
3. The effect of the mixture (size gradation) has not been investigated thoroughly, further research is needed to determine its effect on the mean velocity. The accuracy of the authors' method depends conside ably upon the size of the bed-material.
4. In order to improve the accuracy of the formula, the C_a-curves show in Fig. 11 and Fig. 12 should be classified more accurately accordin to the bed configurations.
5. It is desirable to express the exponents x and y, and the discharge co ficient C_a as a function of a certain dimensionless parameter or parameters. Further research to improve Figs. 6, 9, 10, 11 and 12 is needed.

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APPENDIX

Symbols

		Eq. No.
	Coefficient	(24)
	Constant	(2)
	Constant	(2)
	Chezy's discharge coefficient	(1)
'	Empirical discharge coefficient	(18)
1	Constant	(8)
2	Constant	(8)
3	Constant	(9)
a	Discharge coefficient for alluvial streams	(31)
D	Drag coefficient pertaining to the 50 per cent grain size of the bed material	(26)
	Mean depth of the flow	(19)
o	Diameter of the pipe	(13)
	The 50 per cent grain size of the bed material	(19)
1	Notation of a function	(10)
r	Froude number defined as $\sqrt{\frac{V}{gD}}$ for Flow in very wide channel; for flow in a narrow channel where the side wall effect on the mean velocity is appreciable, its expression is modified as $\sqrt{\frac{V}{gR_b}}$	(24)
	Darcy-Weisbach resistance coefficient	(14)
	Notation of a function	(24)
	Gravitational constant	(19)

h	The value of z at the center of a channel or a pipe
K	$K = \dfrac{\frac{V}{V_*} \frac{\tau_b}{\Delta \gamma_s d} S^{\lambda}}{\left(\frac{R_b}{d}\right)^m F_r^{\,N}}$
k_s	General expression for the height of the boundary roughness
m	Empirical dimensionless exponent pertaining to $\frac{R_b}{d}$
M	Empirical discharge coefficient
N	Empirical dimensionless exponent pertaining to F_r
n	Roughness factor
P	Dimensional parameter
p	Slope of the tangent for the curves shown in Figs. 5 and 8
q	Unit discharge of the flow
R	Hydraulic radius
R_b	Hydraulic radius pertaining the bed
S	Channel slope and also the energy gradient of a uniform flow
U_{max}	Local average velocity at $z = h$
u	Local average velocity along the flow direction at a distance of z from the boundary
V	Mean velocity of the flow
V_*	Shear velocity = $\sqrt{\frac{T_b}{\rho}}$
w	Fall velocity of 50 per cent grain size of the bed material $= \sqrt{\frac{4}{3} \frac{1}{C_D} \frac{\rho_s - \rho}{\rho} gd}$ (assume the grain is spherical)
x	Dimensionless exponent pertaining to the hydraulic radius R_b.
y	Dimensionless exponent pertaining to the energy slope S.
z	Distance from the boundary
z_0	A length parameter depended upon the hydraulic roughness of the boundary
γ	Specific weight of the fluid
$\Delta\gamma_s$	Difference in specific weight between the bed material and the fluid
δ	Dimensionless number
ϵ	Empirical coefficient
η	Shape factor of the grain of the bed material
θ	Empirical coefficient

Karman Universal constant (5)

Empirical dimensionless exponent pertaining to the slope S (24)

Dynamic viscosity of the fluid (19)

Kinematic viscosity of the fluid (8)

Density of the fluid (19)

Standard deviation of the size of the bed material (19)

Local shear at the bed level, $T_b = \gamma DS$ for uniform flow in a very wide channel, and $T_b = \gamma R_b S$ for uniform flow in a narrow channel where the effect of the side-wall is appreciable.

Notation of Function

Dimensional exponent pertaining to Eq. (24) (24)

$\Omega = 0.555$ for plane bed;
$\Omega = 0.565$ for dune bed

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EFFECT OF AQUIFER TURBULENCE ON WELL DRAWDOWN

Joe L. Mogg[1]

ABSTRACT

The change from laminar flow to turbulent flow, in the case of water flow-g through sand, occurs over a wide transition zone beginning with a Reynolds umber of about 10. A method for estimating the exponent of the velocity rm for head loss calculations involving flow into wells is presented. Typi-l calculations show that head losses due to aquifer turbulence may have en over-estimated in previously published material.

These studies were made to determine (1) the effect of turbulent flow in e aquifer on well drawdown, and (2) a simple criterion indicating whether or t turbulent flow is present or may be expected to occur in a given well.

Turbulent flow, as opposed to laminar flow, may be defined as flow under nditions that relate the energy losses to a velocity exponent that is greater an one. In laminar flow, the energy losses are directly proportional to the locity. Most well hydraulics formulas are based on laminar flow con-tions, i.e., doubling the drawdown in an artesian well doubles the yield.

Many investigators have shown that the Reynolds number can be used in rous media to indicate turbulent flow. The usual definition of the Reynolds mber (R) for flow through porous media is:

$$R = \frac{Vd}{\nu} \tag{1}$$

ere V = bulk velocity

d = average grain diameter

ν = kinematic viscosity of water at prevailing ground water temperature

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Field Engr., Edward E. Johnson, Inc., St. Paul, Minn.

The bulk velocity means the porosity of the formation is not considered. Thus the velocity is calculated by dividing the flow rate by the gross area (the formation through which this amount of water is passing.

The average grain diameter (d) has usually been calculated by the follov formula:

$$d = \sqrt[3]{\frac{\sum (N d_g^3)}{\sum N}}$$

Where (d_g) is the arithmetic mean of the openings in any two consecutiv sieves and N is the number of grains of diameter d_g. N may be calculated assuming the grains are all spherical and have the same specific gravity; thus, the number of grains per gram of sand can be calculated, and from tl N is obtained by multiplying the number of grains per gram by the number grams retained between the two consecutive sieves. As can be easily seen this method is laborious.

The kinematic viscosity can easily be obtained from published tables if ground water or test water temperature is known.

Other investigators have indicated that if the Reynolds number is great than 10, turbulent flow rather than laminar flow exists. In some cases, tw (the square) has been arbitrarily chosen as the exponent of the velocity aft the existence of turbulent flow has been established by calculating the Reynolds number to be larger than 10. This means that doubling the flow causes four times as much drawdown or head loss through zones of turbul flow.

The calculation of the Reynolds number would be much simpler if some method of estimating the average grain diameter, other than by use of the above formula, were available. A method, used by the organization with w the writer is connected, is to assume that the average grain size of a wate bearing sand formation is represented by the diameter of sieve upon whicl 70% of the total material is retained. This size has also been used to describe the sample as to coarseness and was selected because it usually re sents the portion of the sieve analysis curve that has the steepest slope. though this assumption is solely empirical in nature, the magnitude of the Reynolds number is, in part, defined by its method of calculation and, ther fore, (d) could be chosen by any reasonable method. The simpler empiric definition of average grain size has been used in the investigation reporte this paper and the magnitudes of the Reynolds numbers obtained will corre spond with it.

To determine the Reynolds number at which the head loss ceases to va with the first power of the velocity (termination of laminar flow) and to as tain the exponent to be used for turbulent flow, three typical graded sands were subjected to laboratory tests. These sands are designated as A, B, C in the accompanying text and illustrations. Sieve analysis curves of the sands are shown in Fig. 1. In addition, data reported by Lockman,(1) pert ing to well-sorted sand and gravels that are normally used in gravel pack of wells, and curves developed by Hudson(2) and Machis(3) were studied.

Each of the tested sands was placed in a constant head permeameter w the flow rate could be varied to obtain Reynolds numbers ranging from wi the laminar flow range to the maximum number obtainable with the equipn Flow rate, head loss, and temperature were carefully measured. The Reynolds number was then calculated by using formula (1) with the averag

rain diameter chosen as the 70% retained size (in inches) as determined rom the sieve analysis curve. The velocity was measured in cm/sec and the inematic viscosity in ft^2/sec. The addition of a constant for dimensional omogeneity resulted in the following equation:

$$R = \frac{2.74 \times 10^{-3} \times V \times d}{\nu} \quad (3)$$

After the data were tabulated, a graph of the hydraulic gradient vs. eynolds number was constructed on logarithmic paper for each of the three ands used in the experiments.

'est of Sample A

The average grain diameter of this sample, as represented by the 70% re- ained size, was 0.055 inches. It is a typical coarse sand and fine gravel ormation. The sample was tested under hydraulic gradients ranging from a

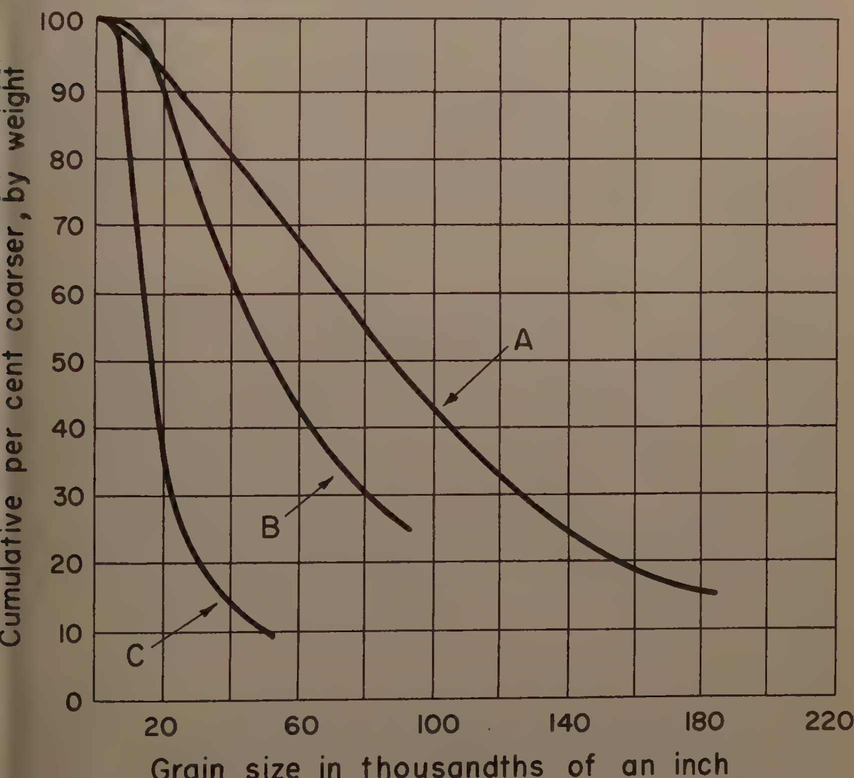


Figure 1. Sieve analysis curves of sands and gravels tested in the laboratory (Samples denoted by letters A, B, and C).

minimum of 0.15 to a maximum of 8.6. The calculated Reynolds number ranged from about 1 to 39. Fig. 2 shows the logarithmic graph of the hydra gradient vs. the Reynolds number. The slope of the first part of the curve unity. This indicates laminar flow, since the hydraulic gradient is proporti al to the first power of the velocity. (Note that in flow through a given sam of sand and gravel the Reynolds number varied only with changes in velocit

At Reynolds numbers larger than 10, there is a definite departure from initial straight line. The slope of the upper branch of the curve is about 1. which indicates that in this range the hydraulic gradient is proportional to 1.2 power of the velocity. The last four plotted points fall slightly above th curve which may indicate that the slope would be greater at higher Reynold numbers.

Test of Sample B

The average grain diameter (70% size) of this coarse sand sample is 0.033 inches. Water was passed through this material under hydraulic gra dients ranging from 0.4 to 7.0. In this experiment, the calculated Reynolds number ranged from 0.4 to 24.5. Fig. 3 shows a logarithmic graph on whic the hydraulic gradient vs. the Reynolds number for two separate runs is pl ted. The slope of the first part of each curve is unity. The curves are es- sentially straight until a Reynolds number of about 10 is reached, beyond which the slopes become steeper (about 1.2) indicating that the gradient (o head loss) varies with the 1.2 power of the velocity.

Test of Sample C

This sample has an average grain diameter (70% size) of 0.011 inches. is classified as a medium sand. The sample was tested under hydraulic g dients ranging from 1.1 to 36, and Reynolds numbers ranging from 0.2 to about 6. A logarithmic graph of the hydraulic gradient vs. the Reynolds number for two separate runs is shown in Fig. 4. This diagram shows two lines, roughly parallel, one for the first run and one for the second run. T later data were judged to be more accurate as some air bubbles were obse in the test apparatus during the first series of readings. This would tend increase the slope as indicated by the plotted data. The slope indicated by curve of test run No. 2 is close to unity, indicating that the gradient was di rectly proportional to the velocity throughout the test. This result is to be expected since the maximum value of the Reynolds number attained in this experiment was about 6.

Lockman's Studies

The experiments conducted by Lockman(1) were made for the purpose testing sand movement into gravel packs surrounding well casings. The d collected, however, enabled the calculation of head losses, velocities, and Reynolds numbers for certain materials. These derived values were anal and the results used to examine the effect of turbulent flow on head losses

(a) A very uniform sand, all passing the 40-mesh and all retained on th 60-mesh sieve, was tested under four different hydraulic gradients. These gradients, when plotted versus the Reynolds numbers on loga mic paper, showed a straight line relationship with a slope of unity. The Reynolds numbers ranged from 0.3 to 2.0.

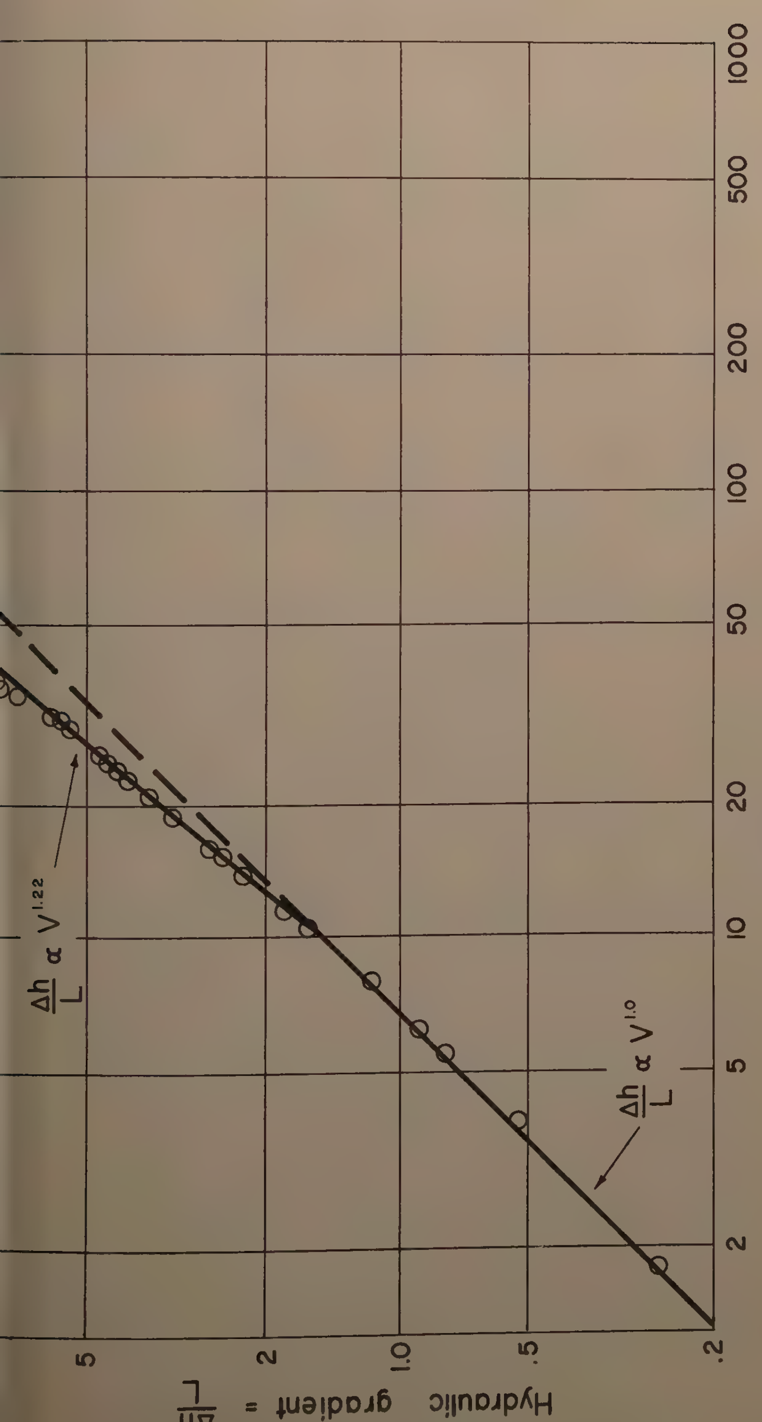
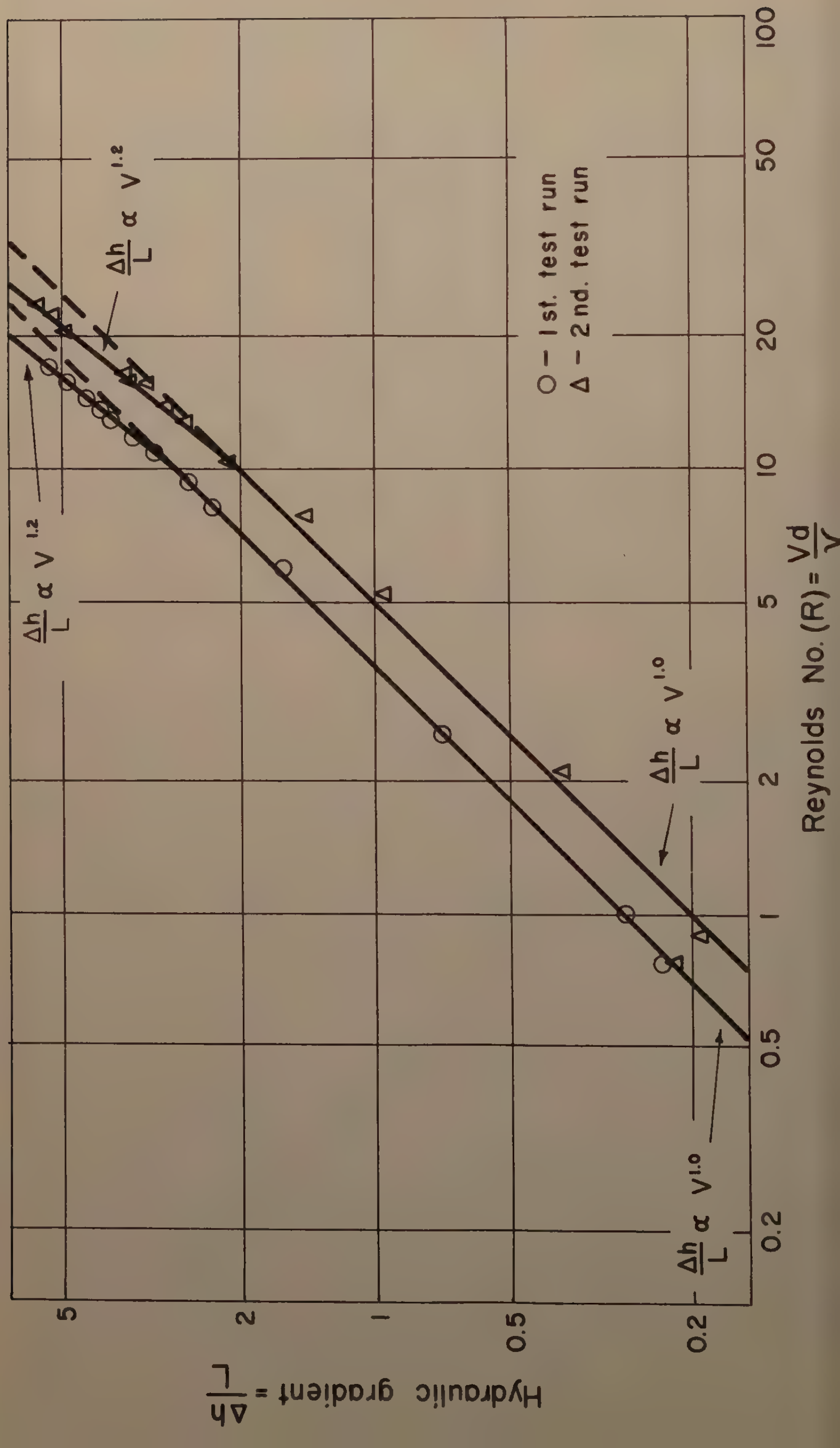


Figure 2. The relationship of the hydraulic gradient and the Reynolds number for Sample A.



$\frac{\Delta h}{L} \propto V^{1.2}$
$\frac{\Delta h}{L} \propto V^{1.2}$
$\frac{\Delta h}{L} \propto V^{1.0}$
$\frac{\Delta h}{L} \propto V^{1.0}$
O – 1st. test run
Δ – 2nd. test run
Reynolds No. (R) = $\frac{Vd}{\nu}$
Hydraulic gradient = $\frac{\Delta h}{L}$
0.2
0.5
1
2
5
10
20
50
100
0.2
0.5
1
2
5

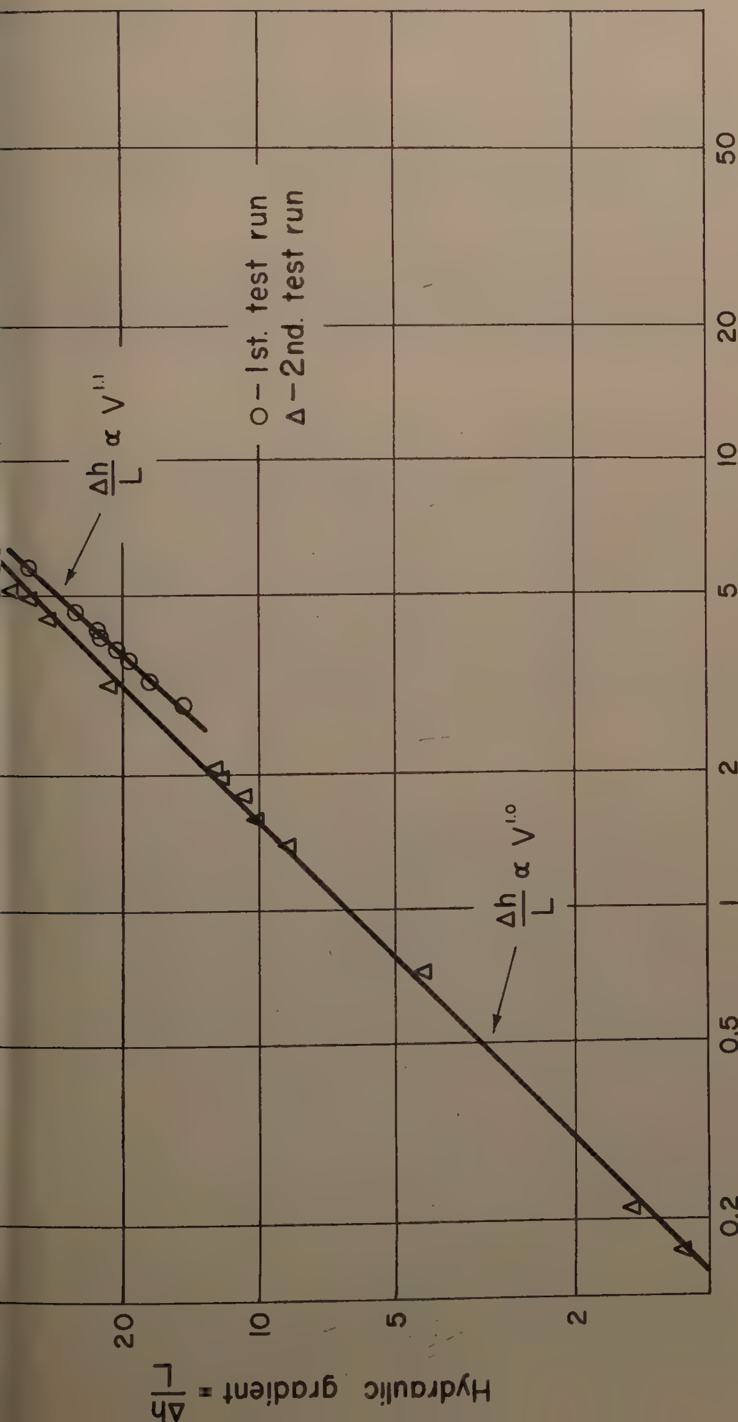


Figure 4. The relationship of the hydraulic gradient and the Reynolds number for Sample C.

(b) A 1/4-inch gravel tested under four different hydraulic gradients showed a variation of the gradient with the velocity to the 1.3 power The Reynolds numbers ranged from 6 to 46 in this series of tests.

A sample of similar material was tested in the laboratory maintaine by the writer's company. This material had an average grain diame (70% size) of 0.208 inches—comparable to the 0.203 inch material u by Lockman. The hydraulic gradient appeared to vary with exponen of the velocity ranging from 1.0 to about 1.3, through a range of Reynolds numbers from 12 to 133.

(c) A 1/2-inch gravel tested under four hydraulic gradients by Lockma showed the possibility of two curves. The hydraulic gradient varied with the 1.1 power of the velocity in a range of Reynolds numbers fr about 12 to 46 and with the 1.6 power of the velocity up to a Reynold number of 92.

Machis's Curve

Machis(3) constructed a graph from the results of testing six different sizes of sands. In order that the different sizes could be directly compar the friction factor versus the Reynolds number was plotted on logarithmic paper. The Reynolds number ranged from 0.1 to 100. Average grain size were calculated by formula (2) thus the magnitude of the Reynolds number slightly different from the values computed for the tests described above. The curve obtained by Machis has a slope of unity until a Reynolds numbe 10 is reached. Between Reynolds numbers of 10 and 100, the slope of the curve changes constantly indicating a transition to turbulent flow. At a Reynolds number of 100, the slope of the curve shows that the hydraulic gradient is proportional to the 1.7 power of the velocity.

Hudson's Curve

Hudson(2) constructed a graph showing the relationship between the fri factor and the Reynolds number. This graph contains data from five sepa investigations and, therefore, offers broader coverage than any of the cur previously discussed. The range of Reynolds numbers is from about 0.1 t over 1,000. The initial slope of the curve is unity. Beginning with a value the Reynolds number of about 4, the slope gradually decreases until a slop about 0.4 is obtained at a Reynolds number of 100. (This indicates that th hydraulic gradient is proportional to the 1.6 power of the velocity.) At a Reynolds number of 1,000 the slope is about 0.2. This indicates that the r lationship of head loss to velocity varies through a wide transition zone wi the exponent of the velocity approaching 2 as a limit for large values of th Reynolds number.

Conclusions from Analysis of Test Data

When all of the test data compiled in the laboratory of the writer's com are compared to the results obtained by other investigators, it appears th Reynolds number of 10 (as calculated in this paper) may correctly repres the limiting point between strictly laminar flow and the beginning of the tr sition to turbulent flow. It furthermore appears that the hydraulic gradie varies with some power of the velocity that gradually increases from 1 to as the Reynolds number increases from 10 to infinity. Hudson's curve sh

at the exponent of the velocity is still increasing at a Reynolds number of 000—although the increase is at a lower rate. Therefore, it appears that the ansition zone, from strictly laminar flow to the development of complete rbulence, is a wide one and that rarely ever (at least in well problems) will omplete turbulence exist.

For practical engineering analysis of well problems, the foregoing data dicate that laminar flow in porous media occurs when physical conditions re such that the Reynolds number is 10 or less. Under these conditions the ead loss varies directly with the velocity. If the Reynolds number is above), conditions are transitional between laminar flow and complete turbulence. ere the head loss varies with the velocity raised to a power between 1 and 2.

To simplify calculations and at the same time introduce a reasonable factor safety, a table was constructed for the purpose of selecting the power of e velocity that the head loss may vary with when the Reynolds number is nown. The table is as follows:

Reynolds Number	Velocity Exponent
less than 10	1.0
between 10 and 20	1.1
between 20 and 30	1.2
between 30 and 40	1.3
between 40 and 50	1.4
between 50 and 60	1.5
between 60 and 70	1.6
between 70 and 80	1.7
between 80 and 90	1.8
between 90 and 100	1.9
over 100	2.0

Conditions of low around and into a well where the Reynolds number may e as large as 100 are rarely encountered.

he Calculation of the Reynolds Number and Its Use in Actual Field Practice

The area immediately surrounding a pumping well is the critical zone for rbulence because of the higher velocities of flow attained in that region. he highest velocity is attained at the point where the water enters a well creen, or where it enters the bore hole in the case of a well drilled in rock. his velocity may be calculated by dividing the pumping rate by the total area f the screen or the area of open hole exposed to the aquifer. The average rain diameter is taken from the cumulative percentage curve of the sieve alysis at the point where 70% of the sand, by weight, is coarser and 30% is ner.

Assume a 12-inch screened well completed in a formation 10-feet thick, omposed of coarse sand similar to sample A. The yield of the well would ave to be 370 gpm, in order to develop flow conditions corresponding to a eynolds number as great as 10 at the well face. To attain a Reynolds number f 100, the yield would have to be 3,700 gpm—a highly improbable figure for e assumed conditions. The effect of turbulence would be represented by a elocity term with an exponent slightly larger than unity for pumping rates reater than 370 gpm, but almost certainly never as large as 2. Also, it must e remembered that the increase in head loss due to turbulence occurs only ithin a short radial distance from the well. If the well in the above example ere pumping 1,000 gpm, the Reynolds number at the well face would be 27,

but at a distance of 11 inches from the face it would be less than 10. Fig. shows a graphical picture of a section of the cone of depression for this we The graph is drawn on semi-logarithmic paper since the head loss (or dra down) is proportional to the logarithm of the reciprocal of the horizontal distance from the center of the well. Therefore, when drawn on this type c chart, the curve representing the cone of depression is a straight line.

The zone containing some turbulence extends 17 inches from the center the well—11 inches from the face of the 12-inch well. The cone of depress from that point inward is distorted to the extent shown by the difference be tween the laminar-flow dashed line and the turbulent-flow solid line. The creased head loss due to turbulence is only 1.4 feet, based on the use of ar exponent of 1.2 for the velocity. The extra head loss due to turbulence rep sents an increase of about 2% over the drawdown calculated on the basis o laminar flow conditions.

This example assumes no increase in permeability resulting from deve ment. Inasmuch as there is some development in almost every well, its e should be considered. As an example, assume that the permeability withir inches of the well face is increased three times by development. The aver grain size in the developed zone will also be increased. It will almost dou at the well face and the resulting Reynolds number will be about 54 insteac 27. However, because this is less than 100, the head loss is still proporti to an exponent of the velocity which is less than 2.0. Assuming that it is 1 the dotted line in Fig. 5 represents the profile of the actual cone of depres after both the effects of higher permeability and higher Reynolds numbers have been accounted for in the developed zone. The total incremental dra down in the zone of turbulence is 3.4 feet. Of this, 1.1 feet is attributable the turbulent flow effects as 2.3 feet would have resulted from laminar flo throughout the zone. When this 1.1 feet which would be caused by the tota drawdown outside the well is compared to the total drawdown outside the v (66.9 feet), it is seen to be an increase of only 1.6 per cent. It is obvious in this well, turbulence, even though it exists, has little effect on the drawdown-yield relationship.

Because Sample B is finer grained than Sample A, and thus has a lowe Reynolds number at any corresponding pumping rate due to its lower aver grain diameter, a yield of 620 gpm would have to be obtained before the Reynolds number exceeds 10 at the well face. Assuming a well with the s physical conditions as Sample A, the effects of turbulent flow would be eve less.

If a well with the same physical dimensions were drilled in a formatio similar to Sample C, a yield of over 1,700 gpm would have to be obtained reach a Reynolds number of 10 at the well face. Even if a 4-inch well wer constructed, the yield would be 400 gpm. Thus, it is unlikely that turbule would be present in such a formation no matter what the physical dimensi of the well. If it were possible to obtain 1,700 gpm from a 12-inch well in this material, the drawdown would be in the neighborhood of 700 feet, an impractical operating condition.

In coarse materials, such as the 1/4-inch gravel discussed in the repo Lockman, turbulence and the Reynolds number are more significant. At a flow of only 98 gpm, the Reynolds number would be 10 at the well face of inch well with a well screen 10-feet long. For a yield of 1,000 gpm, the Reynolds number would be 100 at the well face and less than 10 at a dista of 54 inches from the face. Fig. 6 is a semi-logarithmic profile of the co

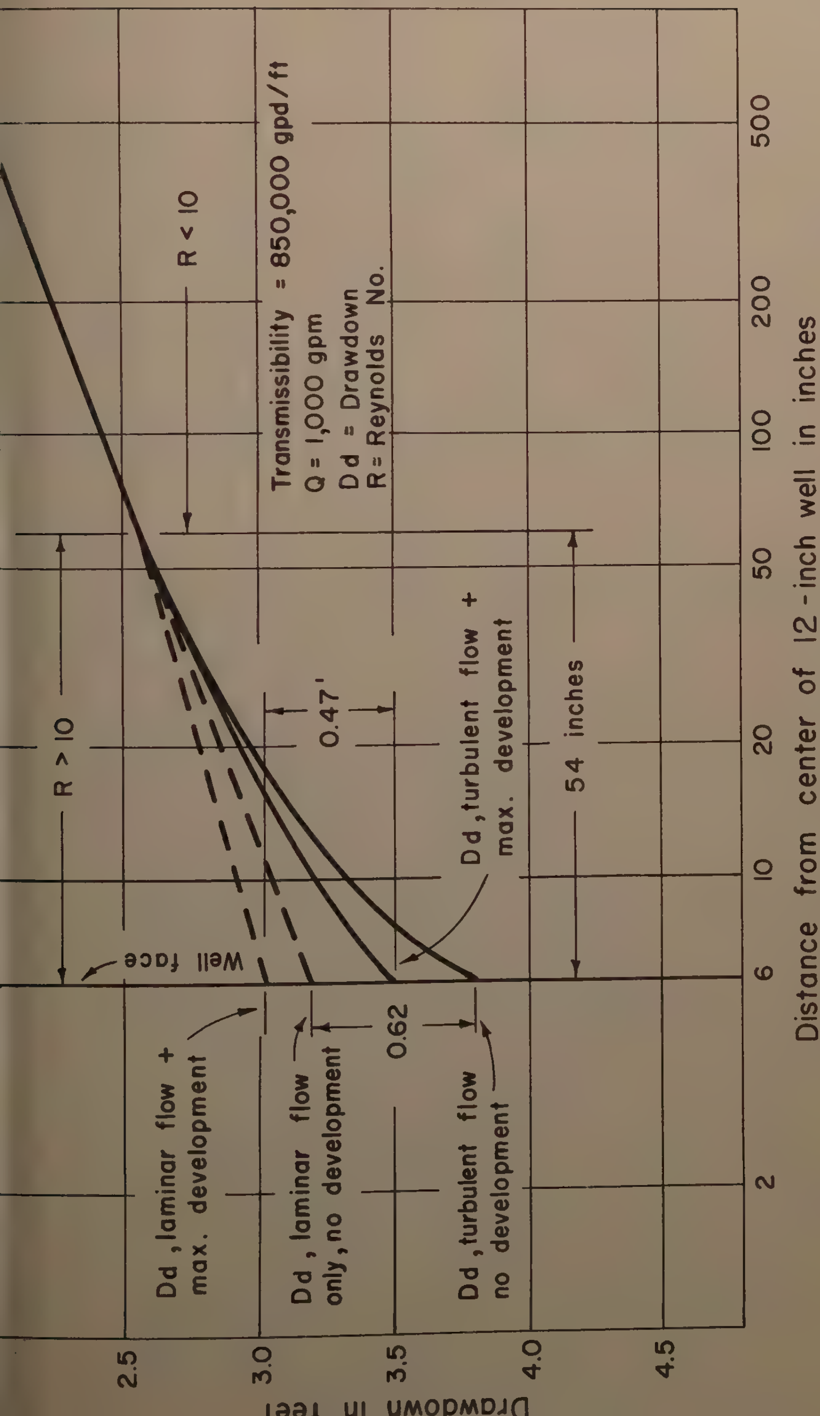
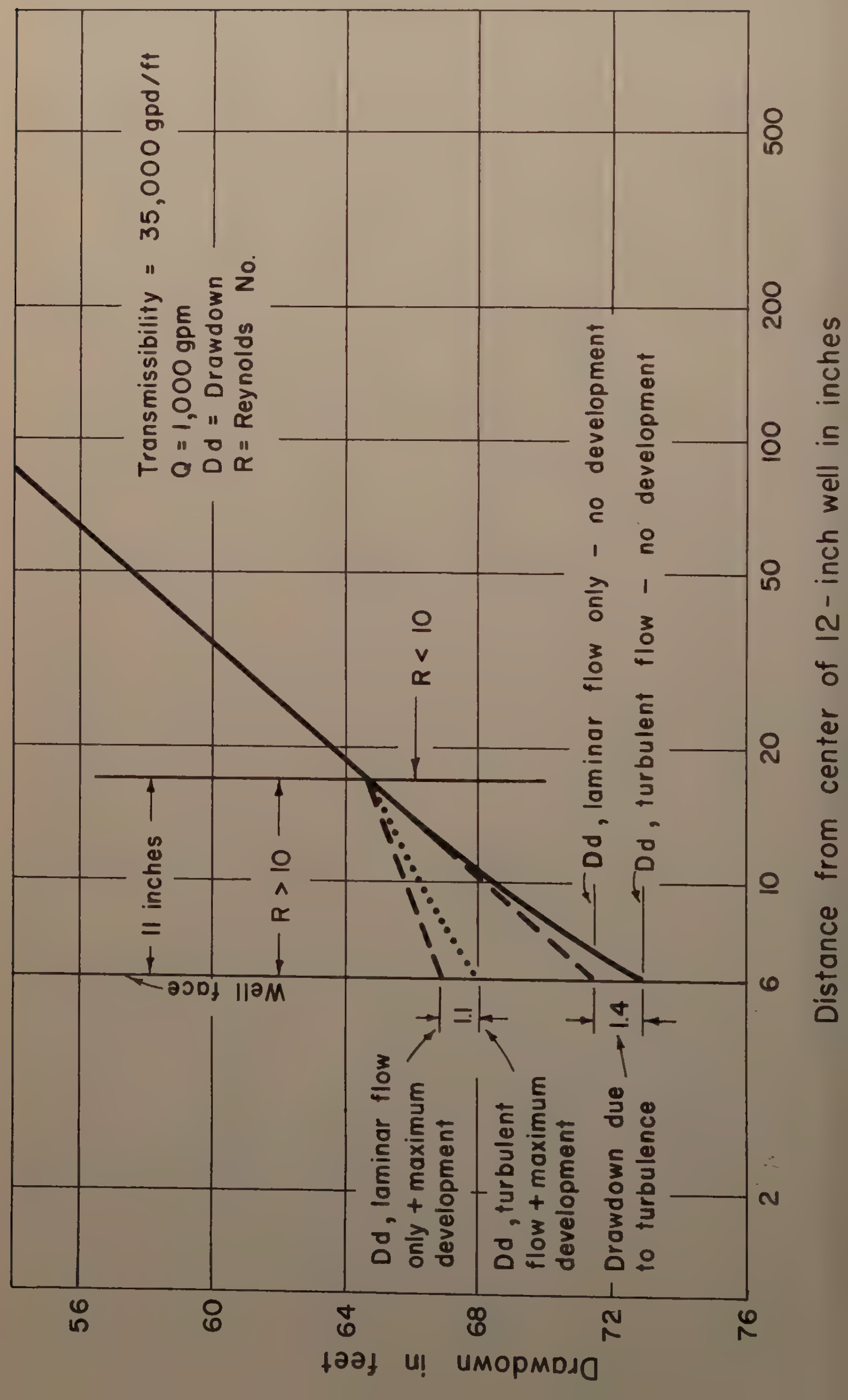


Figure 6. Profile of the cone of depression in a 1/4 inch gravel formation


Transmissibility = 35,000 gpd/ft
Q = 1,000 gpm
Dd = Drawdown
R = Reynolds No.
11 inches
R > 10
R < 10
Well face
Dd, laminar flow only - no development
Dd, turbulent flow - no development
Dd, laminar flow only + maximum development
Dd, turbulent flow + maximum development
Drawdown due to turbulence
1.1
1.4
56
60
64
68
72
76
Drawdown in feet
2
6
10
20
50
100
200
500
Distance from center of 12-inch well in inches

depression of a 12-inch well constructed in 10 feet of 1/4-inch gravel formation. The average grain diameter is 0.208 inches. When the effects of turbulence are considered without development, the drawdown is increased 0.62 feet above what conditions would be with laminar flow. This represents 20% of the total drawdown in the well and should be recognized as a major factor. In this case, it is estimated that the average head loss will vary with the second power of the velocity. This power was chosen because it represents the maximum loss conditions at the well face where the Reynolds number is equal to 100. Actually the average head loss would vary with some lesser power of the velocity because the exponent of the velocity is increasing from 1.0 to 2.0 in the transition zone. By choosing the exponent from the Reynolds number at the well face a second safety factor is automatically incorporated.

Suppose, on the other hand, that development reduces the head loss in the zone of turbulence by 25% and that the Reynolds number is not increased because the average grain diameter is increased only slightly. In our case, the drawdown is only increased 0.47 feet over that which would occur under laminar flow conditions and represents about 13% of the total drawdown in the well.

CONCLUSIONS

It appears that whether or not turbulence exists, or may be expected to exist, can easily be determined by the Reynolds number criteria described in this paper. If the Reynolds number is 10 or less, the head loss varies with the first power of the velocity and laminar-flow well formulas apply. If the calculated Reynolds number is between 10 and 100, the exponent of the velocity can be selected between 1.0 and 2.0 respectively. It is suggested that an exponent be selected by determining the Reynolds number at the well face (highest value) and then referring to the foregoing table. From a profile of the cone of depression the actual head losses can be calculated.

The test data indicate that most well problems concerning turbulence will be found in highly permeable thin aquifers that are being heavily pumped. Even in a formation consisting of 1/4-inch clean gravel (Permeability = 85,000 gpd/ft^2) the contributions of turbulence to the total drawdown were only 13%. Therefore, it seems unlikely that turbulence in the formation adjacent to the bore hole will alter the drawdown-yield relationships in most water wells.

An important factor that appears to reduce the effect of turbulence is maximum development of the formation materials adjacent to the well. It has been shown that suitable development of a well in a 1/4-inch gravel formation could reduce the additional drawdown caused by turbulence from 20% to 13% of the total drawdown.

Other ground water investigators have indicated great losses attributable to turbulent flow effects. However, their measure of the total head losses also includes the friction losses caused as the water passes through the well face (screen); the friction losses caused by the 90° change in direction of the water as it turns upward inside the screen; and the friction losses caused by the upward movement of water through the well casing. All of these losses probably vary with the square of the water velocity. Because the total losses are actually present, as shown from many field pumping tests and this paper indicates that only minor turbulent losses are present in the aquifer immediately surrounding the well, it appears that some of the above losses are more significant.

The friction losses caused by the upward movement of water in the casi can be readily calculated. They are, for the most part, minor losses becau of the short length of water travel.

The screen losses are more difficult to calculate. However, it appears the per cent of open area in the well screen is more important to high yiel ing wells than previously recognized. If the well screen has insufficient op area, not only are high friction losses caused as the water passes through screen, but even more pronounced may be the losses within a short horizo distance from the well face. This is because the water must converge to t limited number of openings in the screen and thus the gross area of flow w in the formation is reduced which would make the calculated velocity and resulting Reynolds number lower than what actually exists.

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Journal of the

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Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

(Over)

ote: Paper 2269 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, HY 11, November, 1959.

WAVE FORCES ON SUBMERGED STRUCTURES[a]

Closure by Ernest F. Brater, John S. McNown and Leslie D. Stair

ERNEST F. BRATER,[1] F. ASCE, JOHN S. McNOWN,[2] F. ASCE and ESLIE D. STAIR.[3]—The authors wish to express their appreciation for the ontributions of the discussors. The value of the paper has been increased rough corroboration, addition, and correction.

Mr. Alterman presented the results of an unusual field investigation which dicated that forces on piling can be accurately determined by means of the asic equations. He also presented formulas for predicting the force on a orizontal cylinder. However, these are for a different type of motion than at discussed in the paper. In accordance with Lamb's presentation,(32) the rce is caused by a uniform flow past a cylinder whose radius is small comared to its submergence. The resistance is calculable from the rate at hich energy is supplied in the form of a surface wave downstream from the linder. The derivation holds for very deep water, and does not include the pecial case of the cylinder resting on the bed which Mr. Alterman presented. urthermore, as stated in the reference, the force becomes zero in the limitg case discussed by Mr. Alterman for which $C = \sqrt{gd}$, (d is the depth of ater, assumed in the derivation to be very large.) The result is related to e authors' study only insofar as the motion within a wave approximates a eady flow, and only for that portion of the resistance which is caused by the eation of a secondary system of surface waves.

Mr. Hamlin called attention to the favorable agreement of the experimental ass coefficients with the theoretical ones and with those determined by u(19) for the barge-like models. The question of correspondence of coefcients and the comment with regard to the frequency of the oscillations in e tests by Yu(19) are related to information which has appeared in several the references in the paper,(5,6) in the discussion by Sarpkaya, and in a ore recent summary of the effects of unsteadiness by McNown and eulegan.(30) Mr. Hamlin is indeed correct in pointing out that the periods in u's experiments were longer rather than shorter (as stated in the paper) an those in the Michigan tests. The statements concerning frequency should ave made use of relative frequency with the reference being the frequency of rtex formation. Presumably, the amplitudes in the experiment by Yu were

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Proc. Paper 1833, November, 1958, by Ernest F. Brater, John S. McNown and Leslie D. Stair.

Prof. of Hyd. Eng., Univ. of Mich., Ann Arbor, Mich.
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Res. Engr., Boeing Airplane Corp., Seattle, Wash., (formerly Assoc. Res. Engr., Eng. Res. Inst., Ann Arbor, Mich.)

so small that no separation occurred in spite of the long periods. The val in the tests of barges and flat plates were such that separation and vortex formation definitely played a role.

Mr. Sarpkaya contributed interesting comments as well as new informa The transverse waves could probably have been avoided by the method he gested. Possibly, however, it was simpler to avoid the occurrence than to eliminate it.

Definitions of virtual and added mass can be presented or defined from several points of view all leading to the same result. The kinetic energy the flow—and its time derivative—are customarily used, and the resulting derivation is entirely logical. Darwin's concept of displacement is an intr ing addition. Another form has been presented by Lamb,[37] in which the integral of the Euler equation, including the term for unsteady flow, is use Still another approach entails the integration throughout the fluid of the in mental force which is the product of an elementary mass and the local acc ation,

$$F' = \iiint \rho \frac{d\bar{u}}{dt} \, dxdydz = M' \frac{dU}{dt}$$

in which $\bar{u}$ is the velocity at any point and U the velocity of the object.

Mr. Sarpkaya's extensive results for separated and composite bodies p vide additional information which is useful. His deductions concerning te for two-dimensional bodies and their application to three-dimensional for are probably valid although subject to some uncertainty, as he suggests. has correctly pointed out the error in sign in going from Eqs. (4) and (5) Eqs. (6) and (7).

Mr. Herbich has reported on difficulties encountered in conducting wa tests in tanks because of the occurrence of transverse waves similar to t observed by the authors. Perhaps the suggestions made by Mr. Sarpkaya this regard and his references to the work at Grenoble will be helpful.

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SNOWMELT RUNOFF[a]

Closure by J. Harold Zoller and Arno T. Lenz

J. HAROLD ZOLLER,[1] F. ASCE and ARNO T. LENZ,[2] F. ASCE.—The uthors wish to express their thanks to Messrs. Wright, Riggs, and Schaefer or submitting discussions to their paper on Snowmelt Runoff.

The work reported by the subject paper was related to the Big Eau Pleine Basin because private funds were allocated for the study in this specific basin. t is true that many basins could have been found for which more precise meteorological data were available. The interesting circumstance here is hat plausible or at least explainable results were forthcoming even though he available data were transferred beyond normal distances. Prior research n this basin(1) has predicted the total runoff volume to be anticipated during ny year as a function of snow cover, water table elevation and other antecedent conditions. Therefore, the question of quantitative runoff raised by Mr. Riggs was considered by the authors although not reported in the subject paper.

The "additional variable" required by Mr. Riggs to explain and account for he wide divergence of curves in Fig. 8, is the total anticipated runoff. Figs. and 10 of the subject paper are intended to translate the total runoff ordinate for each year into a common base at a melt potential of 7 inches, because he runoff for each year was substantially complete at this melt potential for he period studied.

As Mr. Wright reported in his discussion, the limitation of the techniques resented as a means of forecasting daily discharge rate is the accuracy and vailability of such weather forecast data as air temperature, relative humidity, wind velocity and cloud cover. Only insofar as these data can be forecast reliably is it possible to make any reasonable forecast of discharge. Mr. Wright indicates that the runoff forecast can only be made for one day in dvance based on his study of the records for 3 years. The authors' paper nade no attempt to forecast runoff, but it would be helpful if a forecasting echnique might evolve from analyses such as this.

The authors agree with Messrs. Riggs and Wright that the method is only pplicable about 50% of the time as a result of negligible snow pack, frozen ages, or excessive precipitation during the snowmelt period for some years.

The discussions of Messrs. Riggs and Schaefer were concerned primarily vith the adjustments of runoff data for ice effect. Since the bulk of the spring nowmelt runoff usually occurs in a period of from one to 5 days in the Big

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. Proc. Paper 1834, November, 1958, by J. Harold Zoller and Arno T. Lenz.

. Prof. and Chmn. of Civ. Eng., Univ. of N. H., Durham, N. H.

. Prof. and Chmn. of Civ. Eng., Univ. of Wis., Madison, Wis.

Pleine Basin, the authors were particularly concerned with the terminal da of ice effect. Studies indicated that revisions in the published discharges were advisable and these were made accordingly. The decision to make su a revision is almost entirely a matter of personal judgment since there ar no observations available regarding ice floes, ice dams, and the like. In hi discussion, Mr. Schaefer points out back water effects of ice on some Wisconsin rivers as high as 5.69 feet. The subject paper has not contende that back water effects do not occur, but that they are not likely to occur o the Big Eau Pleine River near Stratford because of the relative absence of channel obstructions. It is interesting to note that the reference papers in cated by Mr. Riggs do not include any ice index values in excess of 3 feet a they substantiate in general the magnitude of ice index value used in the subject paper.

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1. Fuhriman, D. K., "Forecasting Snowmelt Runoff", Thesis, Univ. of Wis. 1952.

EXPERIMENTS ON SELF-AERATED FLOW IN OPEN CHANNELS[a]

Closure by Lorenz G. Straub and Alvin G. Anderson

LORENZ G. STRAUB,[1] F. ASCE and ALVIN G. ANDERSON,[2] M. ASCE.— view of the well-known research on the subject of air entrainment carried at the University of Naples, the discussion of Mr. Viparelli in which he mpared some of their work with that of the present paper will aid in a better derstanding of the problem. There are many facets of the air entrainment enomena in open channels that are still in need of investigation, including e basic question of the exact mechanism by which air is insufflated into the ow. It was hoped to present in the present paper data that would be a first proximation in the design of channels in which air entrainment occurs.

All of the data and conclusions as to the mechanism of transport of enained air were based upon measurements of air concentration for various ows and slopes. The mean depth $\bar{d}$ and transition depth d_T were defined in rms of the air concentration distribution. Knowing the water discharge and e mean depth which implies the area per unit width of water flow, the mean locity may be calculated as was shown in Table II.

The concept of the transition depth d_T defined from the shape of the air ncentration distribution curve provided a means of establishing the depth which the mixture of air and water flowed as an open channel flow subject the usual laws of open channel flow. Now if the channel is hydraulically ugh so that the friction coefficient is independent of the mixture Reynolds mber, the resistance coefficient of the aerated flow should be the same as at for nonaerated flow. This assumption was tested by making experiments the same channel under nonaerated conditions and comparing the results. his comparison showed that for the rough channel used in the present experients the resistance coefficient was essentially the same for both aerated d nonaerated flow when the depth of the aerated flow is defined as d_T. The aphical comparison is shown in Fig. 15.

With this agreement in resistance coefficients it was possible to compute e velocity and depth of nonaerated flow having the same water discharges d slopes as the aerated flows used in the experiments. Figs. 16 and 17 show e ratios of the aerated depths $\bar{d}$, d and d_u to the computed depth for an equilent nonaerated flow d_m, as well as the ratio of the mean velocity for rated flow to the mean velocity of an equivalent nonaerated flow.

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a. Proc. Paper 1890, December, 1958, by Lorenz G. Straub and Alvin G. Anderson.

1. Director, St. Anthony Falls Hydr. Lab., Prof. and Head, Dept. of Civ. Eng., Univ. of Minn., Minneapolis, Minn.

2. Prof. of Hydromechanics, St. Anthony Falls Hydr. Lab., Univ. of Minn., Minneapolis, Minn.

Since the mean velocity of the aerated flow was computed from the wat discharge divided by the mean depth $\bar{d}$ as determined from the air concen tration distribution curve, it appeared desirable to make an independent measurement of the velocity in the aerated flow to compare with the comp value. For this purpose velocity and air concentration measurements we made for several aerated flows, from which the mean velocity of the wate could be determined. The mean water velocity was expressed as

$$\bar{V} = \frac{\Sigma_{o}^{d_u} (1 - C) V \Delta y}{\Sigma_{o}^{d_u} (1 - C) \Delta y}$$

in which V is the local velocity and C is the air concentration at elevation normal to the bed.

The mean velocity for an open channel flow for the same discharge anc slope was computed in the usual way in accordance with Eqs. (17) and (18 just as though no air were entrained. The ratio of this velocity V_m and th mean velocity for the equivalent air entrained flow was determined and pl in Fig. 17, which is reproduced with the new points added to it as Fig. 18. The agreement of velocity ratio based upon velocity measurements and th based upon air concentration measurements is remarkably good.

The computation of the normal nonaerated elements of open channel fl in a particular application forms a base from which the elements pertain to aerated flow depart as a consequence of the entrained air. The data ar curves given in Fig. 12, 16, and 17 form the basis for estimating the dept and mean velocity in a steep channel of equal roughness on which air en-trainment will occur. The value of the Manning n for the experimental ch

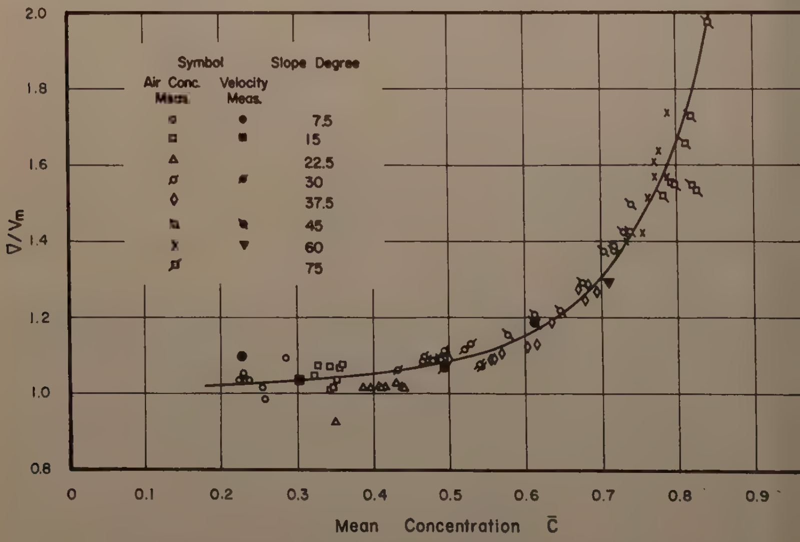

Fig. 18 – Comparison of Mean Velocity of Air Entrained Flow from Velocity Measurements and from Air Concentration Distribution

aries from about 0.012 to 0.016, depending upon the depth of flow, so that e channel roughness corresponds approximately to a surface roughness ach as may be found in practice. In a particular instance of flow on a steep hannel the water discharge and slope are known from which $S/q^{1/5}$, where S the sine of the slope angle, can be computed. The application of this value the curve in Fig. 12 will supply an estimate of the expected mean air oncentration. Entering Figs. 16 and 17 with this mean air concentration will eld the various depths and the mean velocity of the stabilized air entrained ow in terms of the equivalent depth and mean velocity computed in the usual ay as though the flow were not aerated.

RADAR FOR RAINFALL MEASUREMENT AND STORM TRACKING[a]

Closure by Glen E. Stout

GLEN E. STOUT.[1]—The author greatly appreciates the careful review and ditional comments concerning the subject. Since publication of the article, fficulties have been encountered by various agencies in the procurement of Federal Communication Commission allocation for the operation of 10-cm dar for weather purposes. Since 10-cm radar equipment is used by the ederal Aviation Agency and the Air Defense Command for aircraft tracking, is reported that the F.C.C. plans to limit the use of weather radar to the ree and five centimeter wave lengths.

The discussor questioned the use of hourly rainfall data as a comparison th instantaneous echo patterns. It should be pointed out that in this particu-r case the weather system was stationary and there was little movement or ange of echo pattern during the hour. Therefore, it was felt that the use of e echo pattern at the midway point between the hour would be representative the hourly rainfall.

With reference to integrating the signal intensity curve about a cutoff value 65 dbm, the writer questions the significance of this integration. For ex-nple, if P_r is 4.62 decibels higher than the equivalent power from the rain ge, the error is 50 per cent in rainfall amount regardless of the time, nether it is ten minutes or ten hours. On the other hand, the integrated value the difference in powers in decibels with respect to time will vary for fferent storm lengths. It would also appear that a more natural base (cutoff lue) would be the minimum detectable receiver power. The writer sees no ason to believe that this base should in any way be related to attenuation, rticularly in this case where the wave length was ten centimeters.

Proc. Paper 1901, January, 1959, by Glen E. Stout.
State Water Survey Div., Urbana, Ill.

CHANNEL SLOPE FACTOR IN FLOOD-FREQUENCY ANALYSIS[a]

Discussion by William H. Sammons

WILLIAM H. SAMMONS,[1] M. ASCE.—The research reported by Mr. Benson ıs directed toward the development of a statistical technique for determi-ıtion of an optimum main channel-slope factor for use in flood-frequency alyses in New England. This generalized approach may be expected to yield ıproved channel-slope factors and is to be recommended when the nature of e problem does not justify detailed study of channel-slope relationships. ıe paper is extremely stimulating on the history, philosophy and the general tuation of the treatment of hydrologic factors by generalized statistical odels. This presentation is timely and valuable because hydrology as either science or a technology has come of age and the statistical methods have en recognized (demonstrated) as an essential "tool" for the scientist and gineer of tomorrow.

Owing to a general negligence of the problem, the need for an adequate sic-data program in hydrology should be recognized and must be made ıown to all concerned directly or otherwise affected. The author has given some insight into a number of the hydrological parameters entering into e prediction of peak discharges on a generalized basis. His short presen-tion of a few relationships and generalized statements, however, belies the mplexity of the problems involved in dealing with hydrological regression odels.

The development of electronic computers has brought about a misconception at all of the complex mathematical expressions concerned with the behavior the hydrological cycle can be automatically and simultaneously solved mply by pushing a few buttons. Although something of this sort may eventu-lly come to pass, the conversion of the complexities of the hydrological cycle om an art to a work of automatons will require research on the part of a eat many hydrologists, meteorologists, statisticians and engineers for a eat many years. Much theory has yet to be developed. In the absence of sted academic theory, it is customary to set up "models" expressing the eneral characteristics of hydrological situations. A model usually has its ısis in theory that has been "condensed" to certain dominant factors or indi-es that can be evaluated by means of field observations of representative ngth of record, recent maps and surveys, etc.

The author states that "The main-channel slope has been found next in im-ortance to drainage-area size." It is possible, as well as probable, in New

Proc. Paper 1994, April, 1959, by Manual A. Benson.

Hydr. Engr., Soil Conservation Service, Central Technical Unit, U. S. Dept. of Agriculture, Beltsville, Md.

England that "other" hydraulic factors, yet unknown or untried, could als qualify. It is necessary to examine basic assumptions(1,2,3,4,5) of logar regression models as employed by the author. The logarithms of the ori variates were assumed to be transformed to a log-normal probability dis bution. This may be true to a certain extent provided the original variat all have the same or similar probability distribution (characteristics). T latter depends on many items, some are known, others unknown. An exa to illustrate a similar situation but converse to the author's New Englanc ings is presented. A log-regression model between median seasonal yie V_2, (1,000 ac-ft. units) as the dependent variate, median seasonal waters precipitation, P_2, (inches), and drainage area, A, (square miles) as the i pendent variates should illustrate the pertinent facts and modifications. sider the variates tabulated as shown in Table 1-a, "Results of 'First Ap mation' Multiple Regression with Yield", with their respective statistics (first approximation). A graphical examination of the cumulative probab distributions of the original variates indicated the need to rectify the var for normalizing. See Table 2, "Statistics for Original and Transformed ates with Yield". From the statistics tabulated in Table 2, rectification constants(2,6) to normalize the variates were computed by the following:

$$C_i = \bar{X}\,(C_v/CV - 1)$$

where C_i is a rectification constant

C_v is the coefficient of variation of the original variate

CV is the coefficient of variation of the transformed variate

$\bar{X}$ is the arithmetic mean of the original variate

An empirical rectification of V_2, A and P_2 would be $(V_2 + 124)$, (A + 1 and $(P_2 + 68)$ where C_{V2} = 124 (1,000 ac. ft.); C_A = 1172 (sq. mi.); and C_{P2} = 68 (inches). The rectified variates gave reasonable cumulative pr bility distributions when re-plotted on log-normal probability paper.

The new statistics (second approximation) are tabulated in Table 3, "I sults of 'Second Approximation' Multiple Regression with Yield". A com son of the statistics tabulated in Tables 1-a and 3 indicates that a curvil logarithmic model would best represent the data. (The author's Fig. 3 s a similar trend because of the curvilinear regression constants of a etc. For all practical applications the following simplified equation would suf

$$(V_2 + 124) \equiv 1.33 \times 10^{-5}\,(A + 1172)\,(P_2 + 68)^2$$

where A is the drainage area in square miles

P_2 is the median seasonal watershed precipitation in inches

V_2 is the median seasonal yield in 1,000 acre-feet units

Tables 1-a and 3 also indicate that precipitation was more important the drainage area, which is contrary to the author's findings in New Eng Table 1-b, "Results of Multiple Regression with Peak Discharge" confir above statement which was indicated by Tables 1-a and 3. In the latter median annual peak discharge, Q_2 is related to A and P_2 in the same reg (an additional gaging station is included with the peak discharge relation This station lacked runoff data which prevented its inclusion in the volur

Table 1-a. Results of "First Approximation" Multiple Regression with Yield

V_2	Constant and Reg. Coefficients	Standard Errors of Estimate	Correlation Coefficients	"Student"-t	Sample Size
A	3.9575 0.3953	3.2756	0.3045	1.153	15
P_2	9.1135×10^{-3} 2.4287	2.1327	0.7939	4.708	15
A;P_2	5.651×10^{-5} 0.7238 2.8867	1.4249	0.9544	8.261	15

Table 1-b. Results of Multiple Regression with Peak Discharge

Q_2	Constant and Reg. Coefficients	Standard Errors of Estimate	Correlation Coefficients	"Student"-t	Sample Size
A	713.34 0.3189	2.5370	0.3390	1.348	16
P_2	7.4968 1.8321	1.9400	0.7426	4.149	16
A;P_2	7.7068×10^{-2} 0.6035 2.3371	1.0342	0.9596	8.359	16

Table 2. Statistics for Original and Transformed Variates with Yield

	V_2	A	P_2	N
Original Variate (arithmetic units)				15
$\bar{X}$	56.0213	226.5257	29.5333	
S_x	67.4281	272.2752	11.9659	
C_v	1.2036	1.2020	0.4052	
Transformed Variate (log-units)				15
$\bar{X}_\ell$	3.3252	4.9322	3.3035	
$S_{x\ell}$	1.2457	0.9602	0.4072	
CV	0.3746	0.1947	0.1233	

Table 3. Results of "Second Approximation" Multiple Regression with Yield

$V_{2,t}$	Constant and Reg. Coefficients	Standard Error of Estimate	Correlation Coefficients	"Student"-t	Samp Size
A_t	6.9402×10^{-1} 0.7617	1.3213	0.4082	1.612	15
$P_{2,t}$	1.0162×10^{-1} 1.6241	1.2649	0.6378	2.985	15
$A_t; P_{2,t}$	1.3299×10^{-5} 1.0506 1.9187	1.1785	0.8077	3.546	15

Table 4. Statistics for New England and Relation Between Selected Recurre
Intervals and Drainage Area (Peak Discharge)

T-yrs.	Constant a 1/	Reg. Coef. b 2/	Z = ZK*	${}_tR_2$ 3/ $= {}_nR_2 = {}_nR_d$
1.2	22.08	0.880	-0.6375	$0.552M^{0.024}$
2.0	40.00	.856	0.0000	1.000
2 1/3	45.50	.849	+0.1186	$1.1375M^{-0.007}$
5	78.00	.814	.5549	$1.95M^{-0.042}$
10	120	.783	.8448	$3.00M^{-0.073}$
25	197	0.745	1.1539	$4.925M^{-0.111}$
50	282	.717	+1.3536	$7.05M^{-0.139}$
100	398	.690	1.5328	$9.95M^{-0.166}$
150	490	.675	1.6297	$12.25M^{-0.181}$
200	553	.664	1.6975	$13.825M^{-0.192}$
300	670	0.649	+1.7901	$16.75M^{-0.207}$

*ZK will be referred to as Z-values for short.

1. a-values were "adjusted" (smoothed by eye) from a plot on extreme val log-probability paper.

2. b-values were adjusted (computed) from the following -

   $b = 0.88931/T^{0.0551}$ where the standard error of estimate was 1.0088. The b-values for T = 50- and 300-year were omitted as non-representative in the derivation of the above relatio ship.

3. ${}_tR_2$ is the ratio between the magnitudes of a T-event and a 2-year eve

lationship). The location of this generalized example was in a semi-arid gion of the western United States where such results are to be expected.

Because of interactions[7,8,9] between variates, it is very important that possible combinations of the variates be investigated to determine the most icient as well as the most logical order of introduction into the regression odel.

The author's statement ". . . provides the best correlation with flood magnides" is at first misleading until Figs. 1 and 2 are examined and thoroughly derstood. A formal definition of the standard error would be of assistance the readers without a statistical background. The variation of the standard ror with the slope factor is the "key" to understanding the figures.

The term "generalized" versus "regionalized" has been used in the literare with interchangeable meanings. The writer questions which is correct implied in the author's paper.

The author refers to the New England study as a "first phase" and continues state that runoff, precipitation, topographic quadrangle maps and historical od data are available in this area. The writer wishes to emphasize that idies of this type must precede an analysis and be readily available. The eteorological data should be continuous from region to region. It must be of fficient detail to indicate sub-area details, otherwise the variability which ist be explained would be lacking on the maps. An areal integration of the rmal annual precipitation (or runoff) maps should show a normal (or logrmal) probability distribution of the specific variates, otherwise the origil variate should be rectified prior to the introduction into the regression odel. In order to realize the above, the Hydrologic Investigations Atlas, ch as HA-7, should be extended to cover the continental United States (and rts of Canada) so future studies may be realized.

Topographic quadrangle maps of a uniform standard are lacking. Some ates have very recent coverage. The adjacent states (within the hydrological gion) may be in the "horse and buggy" state of mapping. Organization within government agencies and local governments is inadequate to effectively lve this problem. Topographic parameters relate "only" that which has en recorded in our present maps by states and not by hydrological regions. fferent standards of mapping can indicate significance of an index in one gion whereas the same index will fail to show significance in the adjacent gion due to lack of uniform standards for collection of field data. A ogram to fulfill the future needs of hydrological data is desirable.

Historical flood data is indispensable in any regional analysis. The data licates and may give guidance not readily apparent in the raw data. Caution ould be exercised when decisions are made based solely on historical data. e experienced analyst, like a good detective, may wield the historical facts ereas the less experienced individual may only complicate and confuse the uation. A uniform, unbiased method of collection of such data should be a erequisite.

Each variate[5] should be presented as a histogram, frequency polygon or mulative probability distribution. The original (observed) and transformed ctified, etc.) data may be visually compared (the cumulative probability stribution has many advantages over the former). The author deals with nges, etc. which are difficult to visualize.

The statement ". . . the use of mathematically fitted curves involving prior sumptions as to distribution might obscure the very relationships that were ng sought, and that graphically drawn curves, conforming as closely as

possible to the original data at each station would be most desirable" is
weakest link in the regional type investigations, in the opinion of the writ
A priori distribution selected in a haphazard manner or by a policy decis
is very undesirable. Mathematically fitted curves to the "proper" cumul
probability distribution will eliminate much of the bias and errors of intu
tion involved in graphical analysis. Proper rectification (and transforma
of gaging station observed data will eliminate the bias in graphically dra
curves. Plotting of the ordered observed data is of minor importance pr
ed the analyst is consistent and does not incorporate fallacious interpret
to the results. The model selected for the regional study may dictate a
distribution of the variates. The author has used an acceptable plotting
sition formula and should be commended for his choice. The observed d
was plotted on log-normal probability paper. A general concave upward
(as for a in Fig. 3), thus the linear trend of the regression coefficients b
c in Fig. 3 (log-extreme value probability paper). The graphically const
ed curves and the distribution of the variates at selected recurrence inte
creates the concave upward bias of the constant term a on Fig. 3.

Plotting of the constant term, a, and regression coefficient, b, for Q v
leads to linear relationships. (This point will be demonstrated in an exa
to follow in the latter part of this discussion).

Graphically constructed curves dictate the use of several recurrence
vals so as to define the regional relationship over the useful probability
A mathematically fitted curve dealing with the basic statistics[10,11,12,1
the distributions would involve only one, two, or possibly three multiple-
correlations of the variates. The latter approach[10,11,12,13,14] has bee
followed by the writer and many other statistical hydrologists in various
versities and agencies. Limitations of the basic data (observed or origi
tend to support the latter approach rather than the author's refinement.
tematic curvature of the cumulative probability distributions on "Type A
probability paper suggests the use of "Type B" probability paper, thus li
izing the data with the proper transformation.

The author presented eight parameters which were studied. The writ
would like to point out that in all cases where the "average" was used a
substitute of the "median" would prove as good or less variable and mor
satisfactory in a regional analysis. The use of the average rather than t
median can, and usually does, introduce uncontrolled variation which is
difficult to explain or rectify. In summary, the cumulative probability d
bution of each prospective parameter should be known (not assumed). W
ed observed data should be supported or justified before the use of indis
nant weighting coefficients which may only add "insult to injury" in our r
gression model.

The use of topographic maps to obtain elevations and distances for pl
stream profiles depends on the scale ratio. Gaging stations are known p
(elevation) but considerable error can exist in the profile elevations obta
between gages in rugged topography. Small watersheds require toleranc
standards which can not be defined by the present United States Geologic
Survey maps (scale - 1" = 2000 ft. = 7 1/2 minute quadrangles) and must
supplemented by engineering field surveys. Did the author make a comp
son for profiles obtained from different map scale ratios versus a field
survey? If so, what are his recommendations? What difference, if any,
could be noted or expected from the determination of the best slope fact
the Datraton method outlined? An example dealing with a phase of the c
lations for a specific plotted point on Fig. 1 would clarify the technique

mployed (the author indicates that ". . . the '85-10' slope is far simpler to ompute").

The author's Fig. 3, "Variation of Regression Coefficients with Size of lood" is very informative although elusive to the casual reader. The writer ontacted the author[15] and obtained the a and b coefficients for recurrence tervals, T = 1.2, 2.0, 2 1/3, 5, 10, 25, 50, 100, 150, 200 and 300-year events demonstrate one of the many useful relationships that may be developed. dditional information on the hydrology of the region is the result. Table 4, Statistics for New England and Relation Between Selected Recurrence Interals and Drainage Area" tabulates basic regional information.

The a-values plotted as a smooth curve with a concave downward trend on robability paper similar to the author's Fig. 3. The 50-year, a-value plotted bove the curve and the 300-year below. Entries in Table 4, Column 2 are adjusted" a-values from the curve.

The b-values (unadjusted) were plotted on similar probability paper and on g-log paper. The 50- and 300-year events were also non-representative nd were omitted from the computation of the trend line. The log-log plot was tted with the following best fit equation:

$$b = 0.88931/T^{0.0551} \quad \text{with} \quad (\times_{\div}\ 1.0088) \tag{3}$$

here T is the recurrence intervals in years

b is the regression coefficient

1.0088 is the standard error of estimate of the relationship

The only reason for selecting the log-log plot over the extreme value logrobability plot was to simplify the computations. An equation of the latter ould take the following form:

$$b \doteq u\ (1/\alpha)^{\pm y} \tag{4}$$

r the linear form

$$\log b = \log u \pm y \log 1/\alpha \tag{4'}$$

here $\log 1/\alpha$ is the slope

y is the reduced variate employed in the Gumbel distribution and requires a table of y-values for selected recurrence intervals.

ne y-values are not in wide use although tables[14,16] have been published. ne computed b-values from Eq. (3) are tabulated in Table 4, Column 3.

The assumption was made that since a log-regression model was employed the author, the log-normal relationship between recurrence intervals ould apply in the New England region. Columns 4 and 5 were developed om Columns 2 and 3 and from basic relationships of the log-normal probality distribution. From Column 5 of Table 4, Table 5 was computed which bulates tR_2-values for T and A. See Table 5, "Relation Between Selected ecurrence Intervals and Drainage Area." A plot of this table on log-log per gives a graph of expected variation between the magnitude of selected currence intervals in relation to the median (2-year event) and drainage ea.

Table 6, "Relation Between T, C_v and A, New England" was computed from e following basic[1,2,3,4,17,18,19] log-normal probability distribution lationship:

$$t^{R_2} = \log^{-1}\left(Z\left[\log\left(C_v^2 + 1\right)\right]^{0.5}\right)$$

where the general form(3) for any recurrence interval as a base is:

$$t^{R_d} = \frac{\log^{-1}\left(Z_t\left[\log\left(C_v^2 + 1\right)\right]^{0.5}\right)}{\log^{-1}\left(Z_d\left[\log\left(C_v^2 + 1\right)\right]^{0.5}\right)}$$

where Z is an abbreviation for ZK where K is the log-normal critical va
and $Z = \sqrt{\log e}$

A table of ZK-values is published in References 3 and 17.

C_v is the coefficient of variation of the original variate which is dis cussed in References 1, 2, 3, 4 and 17.

Table 5. Relation Between Selected Recurrence Intervals and Drainage Area in New England (Peak Discharge)

T	t^{R_2} -	Values for A at Selected T-values			
Yrs.	1-sq. mi.	10	10^2	10^3	10^4
2 1/3	1.1375	1.119	1.102	1.086	1.066
5	1.9500	1.770	1.607	1.459	1.325
10	3.0000	2.536	2.144	1.812	1.532
25	4.9250	3.815	2.954	2.288	1.772
50	7.0500	5.119	3.717	2.699	1.960
100	9.9500	6.789	4.633	3.161	2.157
200	13.8250	8.885	5.710	3.670	2.359
300	16.7500	10.400	6.457	4.009	2.489

T is the recurrence interval in years.

t^{R_2} is the ratio between the magnitudes of a T-event and a 2-year event.

A is the drainage area in square miles.

The following 5 cumulative probability distribution equations(4,5) of C_V vs are given in lieu of a family of straight lines on log-normal probability paper for A = 1, 10, 100, 1000 and 10,000 square-mile drainage areas:

1 sq. mile:

$$C_v = 1.1465 \pm 0.1901 \text{ K for } C_v = 0.1652 \quad (7)$$

10 sq. miles:

$$C_v = 0.8976 \pm 0.1188 \text{ K for } C_v = 0.1326 \quad (8)$$

100 sq. miles:

$$C_v = 0.6889 \pm 0.0667 \text{ K for } C_v = 0.09637 \quad (9)$$

1000 sq. miles:

$$C_v = 0.5074 \pm 0.0272 \text{ K for } C_v = 0.05328 \quad (10)$$

10,000 sq. miles:

$$C_v = 0.34425 \pm 0.0098 \text{ K for } C_v = 0.02838 \quad (11)$$

Table 6. Relation Between Recurrence Interval, Coefficient of Variation and Drainage Area in New England, Peak Discharge (Based on Basic Data from M. A. Benson

T	C_v for A at Selected T-values				
Yrs	1- Sq. Mi.	10	10^2	10^3	10^4
2 1/3	0.818	0.693	0.578	0.471	0.369
5	0.936	0.764	0.611	0.472	0.343
10	1.041	0.833	0.652	0.490	0.342
25	1.136	0.891	0.683	0.500	0.336
50	1.212	0.939	0.711	0.513	0.337
100	1.286	0.985	0.738	0.527	0.340
200	1.352	1.026	0.762	0.539	0.342
300	1.391	1.050	0.776	0.547	0.345
Average	1.1465	0.8976	0.6889	0.5074	0.3443
Median	1.130	0.895	0.680	0.502	0.343

T is the recurrence interval in years.

C_v is the coefficient of variation of the observed (original) data (Eq.5).

A is the drainage area in square miles.

where the K-values for specific levels of probability must be selected fro table published in References 2, 3, 18 and 19 for the C_v-value given follow each equation.

One of the many ways Table 5 can be applied is to the following equatio employed by the Soil Conservation Service to determine if a "minimum ac ceptable years of record" has been employed in computing a cumulative lo normal probability distribution for a gaging station.

$$y_m = (4.30\ t_{10}\ \log\ {}_{100}R_2)^2 + 6 \qquad ($$

where y_m is the minimum acceptable years of record

t_{10} is "Student's" t at the 10% level of significance, with $(y_m - 6)$ de grees of freedom.

From Eq. (12) and Table 5 the following tabulation was made:

	Drainage Area in Square Miles				
	1 sq. mi.	10	100	1000	10,000
y_m =	57	42	30	20	13-yrs.

Summarizing the implications involved, it could be concluded that if onl drainage area is considered in the New England region to estimate the 2- a 100-year recurrence interval peák discharge, and if the ratios, ${}_{100}R_2$, sho be used to evaluate the expected minimum years of record necessary by co puting y_m from Eq. (12), it then could be concluded that 57 years of record more) would probably be necessary for a gaging station on a 1-square mile drainage area whereas only 13 years will be required for a 10,000 square mile watershed. How many watersheds of 1 square mile have 57 years of record in the New England region? Needless to say, drainage area alone i not sufficient in New England to estimate peak discharge. The addition of t author's slope parameter (and others) should decrease the minimum requi years of record, y_m = 57, to some smaller more logical number for a 1-sq mile watershed.

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TWO METHODS TO COMPUTE WATER-SURFACE PROFILES[a]

Discussion by William C. Peterson

WILLIAM C. PETERSON.[1]—In computations of water-surface profiles by the step method, it is often necessary to adjust a velocity head computed from the average velocity in a subdivided section to a value more representative of the true velocity head. Adjustment is usually effected by the use of an approximation of the correction coefficient

$$\alpha = \frac{\int^{Q} v^2 dQ}{V^2 Q}$$

An approximation of α may be expressed directly in terms of section properties by stipulating (as do the authors) that in a subdivided section

$$Q = K_d S_f^{\frac{1}{2}} \quad \text{and} \quad Q_p = K_{d_p} S_f^{\frac{1}{2}}$$

or

$$V = \frac{K_d S_f^{\frac{1}{2}}}{A} \quad \text{and} \quad V_p = \frac{K_{d_p} S_f^{\frac{1}{2}}}{A_p}$$

then

$$\alpha_1 = \frac{\sum \left(K_{d_p}{}^3 / A_p{}^2 \right)}{K_d{}^3 / A^2} \approx \frac{\int^{Q} v^2 dQ}{V^2 Q} = \alpha$$

With α expressed in terms of section properties, the computation of α_1 can be incorporated in the authors' Table 1 and eliminated from involvement in the trial-and-error step procedures. A revised format of Table 1—from which curves of A, K_d, and α versus elevation can be prepared—is shown in Table 4.

In the authors' Method A the arithmetic mean of the friction gradients at the ends of a two-section reach is used to obtain the friction head, h_f. This use of the arithmetic mean overlooks an interesting relationship, inherent in the continuity equation

a. Proc. Paper 1997, April, 1959, by Joe M. Lara and K. B. Schroeder.

1. Hydr. Engr., U. S. Geological Survey, Los Angeles, Calif.

TABLE 4.--HYDRAULIC CHARACTERISTICS

Section 1

1	2	3	4	5	6	7	8	9
Elevation	n	$\frac{1.486}{n}$	A_p and A	R_p and R	$R_p^{2/3}$ and $R^{2/3}$	K_{d_p} and K_d	$K_{d_p}^3/A_p^2$ and K_d^3/A^2	$\propto$
5701	0.030	49.5	14.5	0.48	0.613	440		1
5702	0.030	49.5	52	1.27	1.173	3,020		1
5707	0.030	49.5	263	4.99	2.920	38,000		1
5711	0.030	49.5	449	7.21	3.732	83,000		1
5713	0.030	49.5	547	8.17	4.057	110,000		1
5713.5-a	0.030	49.5	570	8.55	4.181	118,000	5,057,000,000	
-b	0.050	29.7	57.5	0.25	0.397	678	-	
			627.5			118,678	4,245,000,000	1.1
5714-a	0.030	49.5	594	8.77	4.253	125,000	5,535,500,000	
-b	0.050	29.7	192	0.62	0.727	4,150	1,900,000	
			786			129,150	3,486,900,000	1.5

$$Q_1 = K_{d_1} S_{f_1}^{\frac{1}{2}} = K_{d_2} S_{f_2}^{\frac{1}{2}} = Q_2$$

that becomes evident when the equation is rewritten as

$$Q_{1-2} = (K_{d_1} K_{d_2})^{\frac{1}{2}} (S_{f_1}^{\frac{1}{2}} S_{f_2}^{\frac{1}{2}})^{\frac{1}{2}}$$

or, generally(1)

$$Q = K_{d_g} S_{f_g}^{\frac{1}{2}}$$

That is, Q is a function of the geometric mean of the end-section conveyan and the geometric mean of the square roots of the end-section friction gradients.

Expressing the friction gradient, S_{f_g}, in terms of a "Project Q" and the conveyance, K_{d_g}, leads to a step method of determining water-surface pro- files that requires only the 12 columns of tabular data shown in Table 5. In this method trial values of water-surface elevations are successively as- sumed and the unique value is confirmed when the "Computed Q" equals the "Project Q." Columns 3 to 6 of Table 5 provide data to determine h_f in column 7. The square root of the friction gradient, S_{f_g}, is obtained in colu 9 and used with the conveyance, K_{d_g}, to compute Q in column 12.

The formula for h_f in column 7 follows from the authors' Fig. 1:

$$h_f + M(h_{v_2} - h_{v_1}) + h_{v_1} + d_1 + z_1 = h_{v_2} + d_2 + z_2$$

transposing

$$h_f = (d_2 - d_1 + z_2 - z_1) + (h_{v_2} - h_{v_1}) - M(h_{v_2} - h_{v_1})$$

or

$$h_f = \Delta H + \Delta h_v (1 - M)$$

TABLE 5.--WATER-SURFACE PROFILE COMPUTATIONS

Project Q = 11,100 cfs

1	2	3	4	5	6	7	8	9	10	11	12
tation	Assumed water surface elevation	A	α_1	$h_v = \frac{Q^2\alpha_1}{2gA^2}$	$\Delta h_v = h_{v_{up.}} - h_{v_{dn.}}$	$h_f = \Delta H + \Delta h_v E$	L	$S_{f_g}^{\frac{1}{2}} = \left(\frac{h_f}{L}\right)^{\frac{1}{2}}$	K_d	$K_{d_g} = (K_{d_{up.}} \times K_{d_{dn.}})^{\frac{1}{2}}$	Computed Q = $K_{d_g} S_{f_g}^{\frac{1}{2}}$
ction 1	5714.0	786	1.59	4.93					129,150		
2	5720.9	4533	4.70	.44	-4.49	2.41	500	0.0694	428,000	235,000	16,300
ction 2	?										

here M is a separation-loss (or form-resistance) factor varying from 0.0 to .0. By letting (1-M) = E = an energy-conversion factor, the formula becomes

$$h_f = \Delta H + \Delta h_v E$$

nd is applicable to both contracting and expanding reaches provided complete E = 1.00) conversion of potential head is assumed at all times for contracting eaches. This assumption—equivalent to assigning a value of zero to M for ontracting reaches—appears reasonable when one considers that the total ngle describing a contracting reach is generally much smaller than the maximum angle of 12.5° in converging transition structures for which a value of .1 is usually recommended(2) for M. For expanding reaches (downstream h_v ensibly less than upstream h_v) the value assigned to E may range from 0.0 o 1.0.

As "hydraulic characteristics" were made available only for Section 1 in Table 2 (Method A) of the authors' paper, the sample computations in Table 5 re limited to those for a trial water-surface elevation at Section 2. Needless o say, the assumption of no separation losses in this contracting reach and he use of the weighted conveyance, K_{d_g}, will result in a slightly different water-surface elevation at Section 2 than that produced by the authors' Method A.

Computation of the hydraulic conditions at bridge crossings should take advantage of the wealth of experimental data obtained by Kindsvater,(3) Carter,(3,4) and Tracy,(4) in their studies of open-channel constrictions. The "bridge-crossing" computation would be handled best, perhaps, as a separate nvestigation and the results integrated with those of the step procedure.

The authors' Method B makes provision for a condition where the length of he overbank flow differs materially from the length of the channelized part f the waterway. Undoubtedly, field verification of Method B exists that warrants its use in preference to Method A. The writer would welcome urther evidence of the justification for application of Method B.

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RESISTANCE EXPERIMENTS IN A TRIANGULAR CHANNEL[a]

Discussion by Peter Ackers

PETER ACKERS.[1]—The number of papers published in this journal in recent years on the resistance effects of various types and patterns of roughness shows the great interest of this subject at the present time. The authors' work on large-scale triangular flumes contains very useful information, which has particular significance because the possible effects of the variation of the shape of the flow section with depth are eliminated with an open channel of this form. The writer's interest arises because he has recently carried out roughness experiments on salt-glazed sewer pipes with various eccentricities imposed at their spigot-socket joints; and, contrary to recently-expressed opinion,(1) he found that the Colebrook-White(2) equation (on which the Moody diagram is based) agreed satisfactorily with the experimental results over the full practical range of operation.(3)

Joint spacings of 2 ft and 3 ft were used, and eccentricities varying up to .4 in. were imposed at the joints. The data was used to evaluate ϵ, the equivalent sand roughness in the equation used, and an attempt was made to correlate this with the joint pitch and eccentricity. So as to retain dimensional equivalence, the experimental value of ϵ for each roughness condition was compared with h^2/λ (h being the mean height of the lips of the joints and λ their pitch), and an approximately linear relationship was found to exist between these variables. With this in mind as a possible basis of correlation, the writer therefore attempted an analysis of the data obtained by Powell and Posey, which may be of interest. The authors had reported one quite unexpected result; that without battens their triangular channel appeared to be 'smoother than smooth', and they had to modify the usual smooth-pipe equation for this reason. In consequence, the writer also had to modify the Colebrook-White equation in a similar fashion to

$$\frac{1}{\sqrt{f}} = -2.2. \log \left[\frac{\epsilon}{14.8R} + \frac{2.51}{\mathcal{R}\sqrt{f}}\right]$$

It may well be that the need for changing the factor from 2.0 to 2.2 is a shape effect, arising from the non-uniformity of boundary shear around the perimeter of an open-channel, and the author's opinion on this would be welcome.

Confining attention firstly to tranquil flow (Froude number < 0.7), mean values of ϵ were calculated for each batten spacing, and the suitability of the above equation was then examined by comparing the theoretical velocity with the experimental value. The following results were obtained:

a. Proc. Paper 2108, May, 1959, by R. W. Powell and C. S. Posey.

1. Prin. Scientific Officer, Hydr. Research Station, Wallingford, Berkshire, U. K.

Batten spacing	Average ϵ	% discrepancy in velocity	$\frac{h^2}{\lambda}$
∞	0.0003 ft	3.5	–
24 in.	0.0082 ft	1.4	0.00012 ft
12 in.	0.0216 ft	2.5	0.00024 ft
6 in.	0.0539 ft	3.2	0.00049 ft
3 in.	0.1081 ft	4.5	0.00098 ft

Thus, the percentage discrepancy using the above equation is, for the smooth channel, neither more nor less than the authors obtained with the Manning equation and their modified Prandtl formula. With battens at 24 i spacing, the Colebrook-White equation is a good fit to the experimental dat but it deviates progressively as the isolation of the roughness elements de creases. The implication is that the shape of the transition between smoot turbulent and rough-turbulent flow varies with the batten spacing, but as op channel data does not yield lines of constant relative roughness on the usua friction factor versus Reynolds number diagram, this can not easily be illu trated. However, the transition would probably change progressively from descending curve at high values of pitch to a rising curve, such as occurs with corrugated pipes,[4] with closely spaced battens. It is interesting to o serve that, in spite of the short-comings of the Colebrook-White equation which are apparent with this type of roughness at close spacings, the relati ship between the ϵ and h^2/λ values given above is roughly linear. Howeve it would be unwise to generalise on this basis, as the available data covers one batten height only. Do the authors plan to extend their experiments to clude other roughness dimensions?

The writer has plotted the equivalent roughness against Froude number Fig. 1, confirming the author's report that resistance is higher under supe critical conditions than for tranquil flow. Although there is a great deal of scatter, the plot of the data shows a trend for the apparent roughness to be greatest at Froude numbers of about unity, when surface effects due to cha irregularities would probably be most pronounced.

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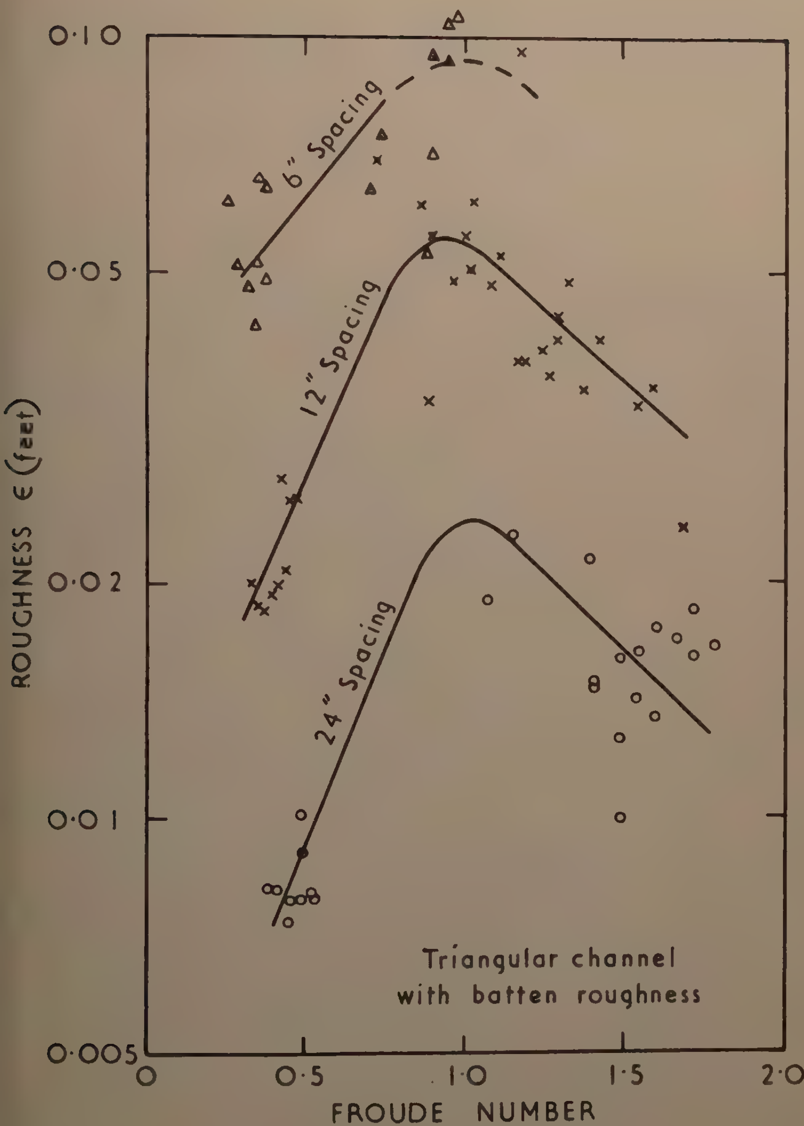

FIG. 1. VARIATION OF ROUGHNESS WITH FROUDE NUMBER (TRIANGULAR CHANNELS)

RESISTANCE PROPERTIES OF SEDIMENT-LADEN STREAMS[a]

Discussion by Emmett M. Laursen

EMMETT M. LAURSEN,[1] M. ASCE.—In this paper the authors very clearly demonstrate the truth of their statement that ". . . when a stream has a movable bed, and sediment is being transported, the problem of determining the resistance is much more complicated than in the simple case of clear water flowing in a channel with fixed walls." On the basis of their observations and measurements they conclude that the interaction of the sediment and the water as it effects the resistance to flow can be considered in two distinct ways. The larger effect is due to the bed configuration (whether ripples, dunes, or flat)—the conditions of flow, and the properties of the fluid and the sediment in some still unknown manner creating a certain bed configuration, and the roughness of this bed configuration affecting the resistance to flow. The observed qualitative relationship between roughness and resistance is entirely in accord with experience with fixed walls, and the magnitude of the effect is not surprising since the dune is much, much greater in size than the sediment particle.

The other and smaller effect observed by the authors is a decrease in the resistance to flow with an increase in sediment transport—all other conditions being presumably unchanged. This effect they ascribe to the damping of the turbulence by the presence of suspended sediment. Certainly, the presence of the suspended sediment must effect the flow and it may well have the effect on resistance suggested by the authors.

However, the writer would like to suggest that this is not yet proved beyond a reasonable doubt since the sediment load could conceivably effect the resistance in another way. Considering a flat bed for simplicity, there is cited a decrease in the friction factor of approximately 25% when a normal sediment load is added to the flow over the fixed granular bed (returning to the conditions of the unfixed bed). In the case of the flow of clear water over the fixed granular bed there is a force transmitted to the flow through the pressure and shear forces on the individual fixed grains. When a sediment load is now added, there will be bed load movement as well as suspended load movement. The resisting force of the fixed grains are now transmitted to the flow at least partly in a different manner. The particles rolling and sliding along the boundary as bed load will be moving at a velocity less than that of the fluid around them in order to receive a propelling force from the fluid. This propelling force will be transmitted to the fixed particles by solid contact. Thus conditions at the boundary and the interaction between the flowing water and

a. Proc. Paper 2020, May, 1959, by Vito A. Vanoni and George N. Nomicos.
1. Associate Prof. of Civ. Eng., Mich. State Univ., East Lansing, Mich.

the fixed boundary should be somewhat different depending on whether or n
the water is transporting a sediment load.

Whether or not the rolling and sliding particles of the bed load moveme
could account for the observed difference in resistance could be checked b
pair of runs such as those reported but with a coarser sand so that the sed
ment would be transported entirely as bed load.

GRAVEL BLANKET REQUIRED TO PREVENT WAVE EROSION[a]

Discussion by P. Bruun

P. BRUUN,[1] F. ASCE.—This article is enlightening and useful to those who ıre trying to introduce better construction practice within this particular ield. Meanwhile, it includes two important assumptions which may not al-vays be easy to keep up with in practice. One is that skilled labor is avail-ble for such construction work which requires great care. The other is the roblem of availability of proper material for a blanket (or rubble mound) esign.

In regard to the first question, it is probably true all over the world that it s more difficult than previously to find skilled labor for such construction vork. One essential reason for this difficulty is the replacement of hand-vork by work done with the aid of mechanized equipment. This makes possi-le the accomplishment of much more work but at the risk of a decrease in ıe requirements of accuracy because connection between the brain and hand f man is closer than the connection between the brain of man and the oper-tion of a mass-producing machine which can never be as flexible as the hand-ool. The question of skilled labor has sometimes become so acute that a ifferent design had to be worked out. Examples of this are the replacements f pitched stone pavements or rubble mound jetties built up of graded layers vith more rugged and less detailed designs. Replacements range from use of nsorted quarry waste material in thick layers under the cover stones (usual-y making the design less stable thus requiring more repairs) to monolithic r block constructions.

In other cases lack of proper filter material—within economic limits— rohibited the use of gravel filters and this circumstance changed the design. 'lorida and the low countries of Europe present many examples of designs of etties, groins and seawalls where the material available was the determining actor in the final design. Attempts have been made to replace gravel filter ıyers and rubble mounds with devices while still preserving some of the good ualities of the normal graded filter layer of traditional type.

Willow-mattresses were used for decades for bottom protection against cour which means as filter layers for special purposes. Nowadays these ave been replaced, among other measures, by asphalt-matresses such as sed in the United States (Mississippi River banks) and Holland (coastal pro-ection and harbor works). The most modern development is represented by ylon sheets which are now being tested (in common use) where the bottom is be protected against scour caused by strong current activity e.g. where aissons for dams, sluices, weirs, and jetties are to be built. The Dutch

Proc. Paper 2021, May, 1959, by Enos J. Carlson.
Head, Coastal Eng. Lab., Univ. of Fla., Gainesville, Fla.

publication on the "Delta Project", July 1959, No. 8 shows how 3 millimet thick nylon sheets can be used. The main difficulty in the application of t sheets still is the technique of bringing the sheets safely to the bottom of channel.

These nylon sheets are entirely impermeable for sand and impermeab is desirable for the work the sheets are supposed to do. Meanwhile the tr definition of a filter is still a layer which, while letting water pass throug holds the material back. It is undoubtedly possible to use permeable shee where the material to be protected is not too fine.

About two years ago a member of the staff of the Coastal Engineering ratory of the University of Florida, J. J. Leendertse, got the idea that Fib glas mat material of insulation type might be useful as replacement for a filter layer in a Florida seawall. The laboratory contracted proper Fiber manufacturers who furnished mat material free of charge for field experi ments with Fiberglas as underlayer for 190 lb. interlocking concrete bloc in a revetment resting directly on a sand slope. A plastic cloth manufact company later asked the laboratory if their plastic woven material would useful in coastal structures. The laboratory found some of its material s able and other field experiments with this material were started at a low elevation (around sea level) than the mat material. Later the proper inte entered into a contract with the laboratory on experiments with the applic of mat as well as cloth material (Fiberglas, plastic mat and woven mater as filter layers in coastal structures including rubble mound jetties, seaw revetments for coastal protection, harbor works, vertical bulkheads, and sheet-pilings. The result of field and laboratory tests which are now bei published by the Engineering and Industrial Experiment Station of the Uni versity of Florida demonstrated definitely that Fiberglas mat material w some improvements now being made is well fitted for application as filte layer above sea level in several coastal structures, particularly revetme bulkheads and seawalls. The mat material was, thanks to its loose struc not applicable below sea level where the cloth material proved successfu particularly in rubble mounds, protective aprons and other bottom protec layers. The same cloth material seems to be useful for tightening leaka in rubble mound jetties (replacing asphalt-grouting) and it may prove use also as protective sublayer in beaches to be periodically nourished artifi with suitable beach material. The sheets should then be well anchored (t down) in the ground. In this way extraordinary losses of material may b prevented and the efficiency of the excellent coastal protection measure artificial nourishment presents may be increased.

The intention of these remarks are in agreement with Carlson's desir stress the importance of well built filter layers of traditional type as we new inventions. Carlson's calling this to attention deserves much credit.

GROUND WATER PROBLEMS IN NEW YORK AND NEW ENGLAND[a]

Discussion by Robert O. Thomas

ROBERT O. THOMAS,[1] F. ASCE.—Ever-increasing demands for additional ater supply development are resulting in a deepened interest and appreciation f the utility of ground water storage for conserving, regulating, and distribut- ig an appreciable portion of the quantities of water required for maintenance f our expanding standard of living. The following discussion stems from a iore or less close association with much of the thought and study for the de- elopment of ground water resources in California.

Conclusions as presented in the paper concisely point out that the existence r occurrence of water in the interstices of the materials composing the rust of the earth is not a separable, independent physical phenomenon but is iextricably linked with the preceding and succeeding phases of the hydrologic ycle. In passing through that cycle from precipitation, through infiltration nd surface runoff, to capture and use, either by nature or by works of man, nd finally to evaporation and return to the atmosphere, to fall again as pre- ipitation, the water droplets many times pass through the state of temporary etention in underground storage. Such a broad concept is philosophically orrect when made without regard to time, whether such be counted in minutes r in eons. Consequently, all subsurface water, even that which has been rapped underground for untold ages, is a part of the common supply and ubject to the natural laws covering hydrologic phenomena. It follows there- rom that man cannot increase the common supply, although by the application f scientific principles to the conservation and use of the available water, he an so manage the available supplies as to increase the benefits to be gained rough the utilization of this constantly renewed, and most important, natural esource.

Utilization of ground water reservoirs for the maximum benefit to the area ivolved is dependent upon adequate geologic and hydrologic data, from which ie results to be expected as a consequence of development can be estimated. 'rerequisite to the full development of ground water storage is adequate echnical information, including the basic data necessary for application of roved hydrologic principles. Required data can be secured by a continuing rogram for measurement of the amount of water passing through each phase f the hydrologic cycle. With adequate hydrologic data, reliable estimates of ie quantity of water that can be made available on a firm annual basis by oordinated or conjunctive operation of surface and subsurface storage or, lternatively, by operation of the ground water basin alone, can be formulated.

. Proc. Paper 2056, June, 1959, by Joseph E. Upson.
. Superv. Hydr. Engr., State Dept. of Water Resources, Sacramento, Calif.

Investigation of physical situations affecting ground water will general be conducted along three broad fronts, or phases, of inquiry. These phas are not characterized by definite boundaries, but rather shade from one i the other in a complex pattern of skills and knowledge. One such area of search may be termed the geological phase, in which the conditions to be countered on and under the surface are determined. The primary requis is the location of underground storage space, which may be found in geol formations that contain inherent voids in sufficient volume to hold, trans and release water in the quantities necessary to satisfy the demand on th supply. Such voids may be solution channels, as in limestone formations brecciated or fault zones as in structural unconformities; or interstices tween the grains composing the formation. The latter type is ordinarily in alluvial valley fills and cones and is composed of gravel or sand, or mixtures of the two, usually interlarded with fine clays or other less per ble material. After determination that usable storage space for practica operation exists, the porosity, transmissibility, chemical composition, lo cation of intake areas, and other basic physical characteristics pertainin hydrologic and hydraulic operations are determined.

The second field of investigation of subsurface storage areas is the hy logic phase. This phase is concerned with determinations of water suppl available for the project; the regimen of its occurrence; the quality of the native, imported, and subsurface waters; the operation of surface faciliti for storing, transporting, and percolating surplus supplies in permeable areas; the recovery of water placed in subsurface storage; and innumera additional factors bearing on the entire problem of routing water supplie tween the place of origin and place of final disposal.

The economic phase is the third broad field of investigation. This pha concerned with the determination of places and amounts of use. It involv the classification of land areas as to suitability for irrigated agriculture domestic, industrial, and other uses; determination of water requiremen the various uses; production, distribution, salvage, and disposal of water plies made available; and other necessary studies.

It should be understood that the above grouping of phases of water sup investigation is made solely for convenience in illustration and bears no lation to activities in various fields of endeavor. The engineer and the g gist will function in close coordination throughout the investigation and w assisted as required by chemists, biologists, physiologists, economists, ists, attorneys, meteorologists, and other concerned specialists.

Effective utilization of the ground water reservoir requires continual measurement of items such as quantity in storage, recharge, and discha and, in addition, the rate of movement of water through the reservoir. T techniques for these measurements are more complex and the fundament information needed to translate the basic data into volumes of water is f more imposing than the requirements for a surface reservoir. Delineati the areal extent and thickness of the rock materials of a ground water re voir is primarily a geologic problem. The quantity of water in those ma is dependent upon the porosity. Permeability is a major factor in the ra movement of water through the reservoir and therefore in the yield of we or springs. The geologic and hydrologic characteristics of aquifers, and lineation of the water table or piezometric surface throughout the ground reservoir are the basis for the quantitative determinations of storage or

movement of water within the reservoir. Thereafter, changes in storage or movement may be calculated from fluctuations of the water table.

Increments to storage also affect the position of the water table in the recharge area. Therefore, coordinated analyses of records of infiltration, soil moisture, storage and discharge, stream seepage, and ground water storage in the recharge area are needed for a thorough determination of ground water recharge. Similarly, determinations of natural discharge from the ground water reservoir require adequate data as to evapo-transpiration, outflow in springs and seepage to streams, loss of storage by subsurface outflow and changes in the volume of water in storage.

Many reservoir problems are chiefly problems of movement of water within an aquifer rather than of replenishment to the aquifer. Developed aquifers serve both as reservoirs to hold water in temporary storage, and as transmission channels to carry water to withdrawal areas from the areas of recharge. The perennial yield of a well or group of wells is determined by the quantity of water that can move through the aquifer from intake area. If the transmissibility of the aquifer is inadequate, the water levels in wells will decline, whether or not the aquifer as a whole is adequately recharged. Problems of inadequate transmissibility of aquifers have developed in every part of the country. Pumping from a well or closely spaced group of wells creates a cone of depression in the water table or pressure surface of an aquifer. Generally, if withdrawal of water continues at a constant rate, the cone expands and the pumped water draws water from a progressively increasing area. The water level in the well continues to drop, but at a decreasing rate, until the cone has expanded to reach either an area of natural recharge or discharge. At such time as the amount withdrawn is balanced by increased movement from the recharge area or decreased natural discharge, the progressive lowering of the water table will cease due to stabilization of rates.

The solution to problems of apparent or local shortages of water is to effect a balance between the rate of draft and the rate of replenishment, either by reducing the draft or increasing the replenishment, or both. The principle is the same as that involved in the elimination of overdraft from reservoirs where long accumulated storage is being progressively depleted. Other corrective measures that are effective in many areas include development of wells in previously untapped portions of the aquifer or redistribution of wells to draw from a more extensive part of the aquifer. Such measures tend to reduce concentrated draft upon a small portion of the aquifer.

This brief resume of the general nature of problems connected with ground water development will, it is hoped, serve to indicate the intricate nature of considerations affecting a given situation. These considerations are frequently of no small moment and successful solutions will contribute to the future development of water resources in the more humid zones of the country.

PRESSURE CHANGES AT OPEN JUNCTIONS IN CONDUITS[a]

Discussion by Fred W. Blaisdell and Philip W. Manson

FRED W. BLAISDELL,[1] F. ASCE and PHILIP W. MANSON.[2]—The authors ave done an excellent job of analyzing and presenting the mass of data which ıey obtained.

The writers conducted tests on square-edged pipe junctions between 1953 nd 1959 and are thus interested in the authors' results even though the ınctions are different in form. In the writers' work, the upstream and down-tream pipes were of the same size and this discussion will be restricted to ı-line pipes of the same size. The writers' laterals covered a wider range f sizes than did the authors'. Whereas the authors tested only a 90° junction, ıe writers' junction angles varied from 15° to 165° in 15° increments. Only ıe 90° junction will be compared with the authors' results. In general, the ame techniques were used by both experimenters.

In spite of the large number of tests performed by each experimenter, a irect comparison of the results is possible only for the one case in which all ıree pipes are of the same size. The in-line loss coefficient agrees very ell with data obtained by the writers and other experimenters for the square ınction, and less well for the rectangular junction box when a large portion f the total flow enters from the lateral. Similar good agreement between the ıthors' data and data obtained by the writers and others is exhibited for the ıteral loss coefficient except when all the flow is from the lateral.

It is reasonable to expect better agreement between the authors' and the riters' results for the square junction than for the rectangular junction be-ause the square junction more closely approximates the closed conduit used y the writers. In view of the rather large difference in the types of junction, ıe agreement of the two sets of data is remarkable.

Simple reasoning would indicate that Eq. (4) which apparently applies so ell to the rectangular junction, should apply to square and round junctions lso. Is there an explanation why different formulas were found necessary? r is the agreement found fortuitous?

Proc. Paper 2057, June, 1959, by William M. Sangster, Horace W. Wood, Ernest T. Smerdon and Herbert G. Bossy.

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T technical papers published in the past year are identified by number below. Technical-division rship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air ort (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hy- s (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil nics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the sym- (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To te papers in the Journals, the symbols after the paper number are followed by a numeral designating ue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 9 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics on during 1958.

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c Discussion of several papers, grouped by divisions.

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